William P. Morse

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXV—No. 1

January 1909

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OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. XXXV—No. 1.

JANUARY, 1909.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

American Society of Civil Engineers

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GEORGE F. SWAIN

Term expires January, 1911:
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EMIL SWENSSON

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays. Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER......................5913 Columbus.
CABLE ADDRESS......................."Ceas, New York."

Vol. XXXV. JANUARY, 1909. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

December 16th, 1908.—The meeting was called to order at 8.30 P. M.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, 148 members, and 29 guests.

A paper by William L. Sibert, M. Am. Soc. C. E., entitled "The Improvement of the Ohio River," was presented by the Secretary, who also read a communication on the subject from Theron M. Ripley, M. Am. Soc. C. E.

The paper was discussed orally by Messrs. G. W. Kittredge, R. B. Stanton, and Thomas C. Atwood.

The Secretary announced the following deaths:

Edmund Louis DuBarry, elected Associate, January 6th, 1875; died December 4th, 1908.

Charles Tarbell Dudley, elected Junior, September 6th, 1904; Associate Member, October 3d, 1906; died September 30th, 1908.

Leslie Monroe Fry, elected Junior, June 6th, 1905; Associate Member, April 3d, 1907; died November 10th, 1908.

Edward Prince, elected Junior, February 6th, 1878; Member, November 1st, 1882; died December 12th, 1908.

Adjourned.

January 6th, 1909.—The meeting was called to order at 8.30 p. m.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, 169 members, and 36 guests.

A paper by George B. Francis, M. Am. Soc. C. E., entitled "Electric Railways in the Ohio Valley Between Steubenville, Ohio, and Vanport, Pennsylvania," was presented by the author, and illustrated with lantern slides.

The paper was discussed by Messrs. F. Lavis, W. J. Boucher, George B. Preston, and the author.

The Secretary announced the election of the following candidates by the Board of Direction on January 5th, 1909:

As Associate Members.

Saint George Henry Cooke, Chester, Pa.
John George Lawrence Cunningham, Williston, N. Dak.
David Leslie Diehl, Harrisburg, Pa.
Byron James Lambert, Iowa City, Iowa.
Edward Percy Lane, New York City.
Hiram Miller, La Paz, Bolivia.
Myron Hall Peck, Tientsin, China.
Francis William Perry, New York City.
Edwin Job Pickwick, Schenectady, N. Y.
George Abel Pierce, Urbana, Ohio.
George Shirley Walter, Chicago, Ill.

As Juniors.

George Simpson Armstrong, Jr., Brooklyn, N. Y.
Joseph Lawrence Brennan, New York City.
Harry Collins Walton, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on January 5th, 1909:

FROM ASSOCIATE MEMBER TO MEMBER.

HERMAN KARL ENDEMANN, Long Island City, N. Y.

FROM JUNIOR TO ASSOCIATE MEMBER.

ALFRED THOMAS BROWN, White Plains, N. Y.
HERBERT WILLARD GODDARD, Boston, Mass.

The Secretary announced the election of the following candidate by the Board of Direction on October 6th, 1908:

AS JUNIOR.

DAVID ADAMS CALHOUN, New York City.

The Secretary announced the following resignations as of December 31st, 1908:

Members: CHARLES MACRITCHIE, THOMAS DELANO WHISTLER.

Associate Members: AUSTIN CURTIN HARPER, CARL LOUIS EDUARD SCHENK.

Associate: WILLIAM FERRIS BOOTH.

Juniors: WILLIS ROBERT JORDAN, WILLIAM HENRY SNYDER.

The Secretary announced the following deaths:

RUTGER BLEECKER GREEN, elected Junior, May 5th, 1896; Associate Member, October 5th, 1898; Member, September 6th, 1904; died December 8th, 1908.

JOHN EGBERT MCCURDY, elected Member, April 1st, 1896; died December 15th, 1908.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract.)

January 5th, 1909.—President Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Churchill, Gibbs, Hazen, Schneider, Smith, Stearns, Swensson, and Tillson.

It was decided that the next Annual Convention of the Society shall be held at the Mount Washington Hotel, Bretton Woods, N. H., July 6th to 9th, 1909.

A report of the Board covering the year ending December 31st, 1908, for presentation to the Annual Meeting, was adopted.

Ballots for Membership were canvassed, resulting in the election of 11 Associate Members and 3 Juniors, and the transfer of 2 Juniors to the grade of Associate Member.*

The Secretary reported the entry of *Transactions* as second-class mail matter, beginning with Vol. LXI, for December, 1908.

Charles Macdonald was appointed to represent the Society on the John Fritz Medal Board of Award for four years.

Action was taken on members in arrears for dues.

The resignations of 2 Members, 2 Associate Members, 1 Associate, and 2 Juniors were received and accepted, taking effect as of December 31st, 1908.†

Applications were considered, and other routine business transacted.

Adjourned.

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* See page 2.
† See page 3.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, February 3d, 1909.—8.30 P. M.—A paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., entitled "The Purification of Ground-Waters Containing Iron and Manganese," will be presented for discussion.

This paper is printed in *Proceedings* for December, 1908.

Wednesday, February 17th, 1909.—8.30 P. M.—At this meeting a paper by R. P. Bolton, M. Am. Soc. C. E., entitled "The Operation of Passenger Elevators," will be presented for discussion.

This paper is printed in *Proceedings* for December, 1908.

Wednesday, March 3d, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Action of Frost on Cement and Cement Mortar, Together with Other Experiments on These Materials," by Messrs. Ernest R. Matthews and James Watson; and "The Bonding of New to Old Concrete," by E. P. Goodrich, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

PAPERS AND DISCUSSIONS

The Board of Direction has decided to issue *Transactions* in future as a quarterly publication. There will therefore be four volumes each year instead of two as heretofore.

There seems to be some misunderstanding about this matter, several members having protested on the ground that the present volumes are not too large, and that making them smaller would make reference to them more difficult, as well as increase the cost of binding.

It seems, therefore, necessary to explain that the decision of the Board is based on the fact that the amount of valuable material offered the Society for publication has so increased of late that a large part of it would have to be refused if only two volumes per annum were issued, and on the belief that the most effective way of increasing the usefulness of the Society to its Members and the Engineering Profession is to increase the volume of its Publications.

The decision was arrived at only after most careful consideration of the matter from every point of view, and it is hoped the result will be satisfactory to the membership.

It is also hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of the paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each must be established. It is expected that the first volume for 1909 will be issued on or about March 31st, the second on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31ST, 1908.

In compliance with the Constitution, the Board of Direction presents its report for the year ending December 31st, 1908.

MEMBERSHIP.

The changes in membership are shown in the following table:

Grade.	Jan. 1st, 1908.			Jan. 1st, 1909.			Losses.				Additions.		Totals.	
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.
Honorary Members	....	8	8	....	8	8	....	....	....	....	....	....	....	....
Corresponding Members	....	2	2	....	2	2	....	....	....	....	....	....	....	....
Members	492	1 726	2 218	525	1 845	2 379	....	4	5	35	*79	117	44	196
Associate Members	385	1 056	1 441	401	1 218	1 619	76	4	6	6	†64	206	92	270
Associates	62	94	156	68	93	161	3	3	2	2	....	15	10	15
Juniors	137	426	563	152	490	642	64	5	7	1	....	156	77	156
Fellows	9	14	23	7	15	22	....	....	....	1	....	....	1	....
Total	1 085	3 326	4 411	1 153	3 671	4 824	143	16	20	45	143	494	224	637

* 76 Associate Members, 1 Associate, and 2 Juniors.
† 2 Associates and 62 Juniors.

The net increase during the year, 413, while not as great as that of 1907, doubtless due to the prevailing financial conditions, is much greater than the increase in any other previous year. It would have been somewhat larger had it not been for the amendment to the Constitution which went into effect in November, and to the changed rules of the Board of Direction, which deferred the election of the candidates on one preliminary list until after December 31st. The average net increase for the past five years has been 380.

The number of applications received during the year was 728: 536 for admission, and 192 for transfer.

The losses by death reported during the year number 45, and are as follows:

Members (35): George Irving Bailey, Oliver Weldon Barnes, Elbridge Harlow Beckler, Walter Gilman Berg, William Beverly Chase, James Dun, John Edwin Earley, Nathaniel Marsh Edwards, George Edwin Evans, Irving Tupper Farnham, Rutger Bleecker Green, John Bradford Harper, Charles Hermany, Wilhelm Hildenbrand, Henry Randolph Holbrook, John Egbert McCurdy, Martin William Mansfield, Aniceto G. Menocal, Thomas C. Meyer, Stephen Arnold

Mitchell, Othniel Foster Nichols, Herbert Franklin Northrup, William Anson Pearson, Jr., Frederic Auten Combs Perrine, Edward Prince, George W. Rafter, Louis Younglove Schermerhorn, Mark William Schofield, George Edward Sleeper, Edward Clinton Terry, George Edward Thomas, Richard Fenwick Thorp, Charles Harold Tutton, Clarence George Vaughn, Frederick Conover Warman.

Associate Members (6): Charles Tarbell Dudley, Leslie Monroe Fry, James Isaac Haycroft, William Seaton, Jr., Joseph Milton Walker, Clinton Glencairn Wells.

Associates (2): Edmund Louis DuBarry, William Roberts.

Juniors (1): Adolph DeHaas.

Fellows (1): Nathaniel S. Bouton.

LIBRARY.

The total contents of the Library, and the increase during the year, are shown in the following statement:

	Total Contents.	Increase during 1908.
Bound volumes....................	17 590	1 161
Unbound volumes..................	35 300	3 564
Specifications	6 633	46
Maps, photographs and drawings....	4 090	209
Total........................	63 613	4 980

Of these accessions, 2 074 were donations received in answer to special requests; 86 were donations from publishers; 2 672 were donations received in regular course, and 148 were purchased.

The value of accessions to the Library during the year is as follows, each accession having been valued separately as received:

4 832 Donations and exchanges (estimated value)	$2 585.21
148 Volumes purchased (cost)..............	389.63
Binding 318 volumes.........................	407.89
Total.................................	$3 382.73

The following amounts have been expended upon the Library during the year:

Purchase of books, subscriptions, express charges, etc..	$416.29
Binding	407.89
Fixtures, supplies, and sundries................	100.83
Total....................................	$925.01

The number of titles in the Library is 23 408.

The total attendance in the Reading Room and Library during the year was 4 285; a considerable increase over previous years.

During the year 74 new bibliographies (containing 2 253 separate references) have been made, copies of 30 searches made in previous years have been furnished, 7 of these having been brought up to date. The total cost of this work, $473.10, has been paid by those for whom it was undertaken.

In November, 1897, the Society moved into the present House, and on December 31st, 1897, the total number of books, pamphlets, specifications, maps, etc., in the Library was about 27 000. In the report of the Board of Direction for that year, attention was called to the general condition of the Library, and the hope was expressed that the new conditions would result in large and valuable additions. Attention was also called to the amount of shelf room provided in the New House, of which it was estimated the contents of the Library would then occupy about one-third. As stated above, the number of accessions has increased until at the present time it is 63 613, and the vacant shelf room referred to eleven years ago will soon be insufficient to take care of the normal growth. In consequence of this, an additional tier of stacks, space for which was wisely provided at that time, has been ordered, and will be installed early in 1909. This will double the present capacity, and will probably be sufficient to take care of the accessions of the next ten or fifteen years. Owing to the enlargement of the House in 1905, there is also available for future use additional space, which will admit of further extension of the shelving whenever it is needed.

PUBLICATIONS.

During the year ten numbers of *Proceedings* have been issued regularly, and two volumes of *Transactions* have appeared.

In *Proceedings,* the list of references to current engineering literature has been continued, and has covered 81 pages, and contained 3 297 classified references to periodicals.

The stock of the various publications of the Society kept on hand for the convenience of members and others now amounts to 143 626 copies, the cost of which to the Society for paper and press work only has been $19 745.14.

In addition to the regular publications, the second volume of the Index to *Transactions,* covering Volumes XLVI to LIX (20 volumes, including the six extra Engineering Congress volumes), was compiled during the year and issued to the membership without extra charge.

During the year, 8 356 Volumes of *Transactions,* and 2 737 copies of the Index to *Transactions,* have been bound for members and others in the standard half-morocco and cloth bindings.

SUMMARY OF PUBLICATIONS FOR 1908.

	Issues.	Average Edition.	Total Pages.	Plates.	Cuts.
Transactions (Volumes).........	2	5 075	1 209	131	233
Proceedings (Monthly Numbers).	10	5 295	1 988	145	268
Constitution and List of Members	1	5 500	346	...	1
Index to *Transactions,* Vols. XLVI to LIX.............	1	5 200	110	...	...
Totals..................	14	....	3 653	276	502

The cost of publications has been:

For Paper, Printing, Binding, etc., *Transactions* and *Proceedings*	$17 769.87
For Plates and Cuts..................................	2 589.94
For Boxes, Mailing Lists, Copyright and Sundry Expenses.	795.35
For 8 000 Extra Copies of Memoirs and Papers..........	1 048.68
For List of Members..................................	1 646.50
For Index to *Transactions,* Vols. XLVI to LIX..........	462.69
Total..	$24 313.03
Deduct amount received from sale of publications.........	3 268.95
Net cost of publications for 1908.................	$21 044.08

A comparison of these figures with those given in the last report of the Board shows that in *Transactions* and *Proceedings* 315 more pages were issued in 1908 than in 1907. This indicates very clearly that the amount of material offered the Society for publication has increased very much of late, and your Board has decided to enlarge the publications. In order to do this, it will be necessary to issue more than two volumes per annum, because a volume of more than 600 pages is unwieldy, unsightly and inexpedient. The *Transactions,* therefore, will be issued hereafter as a quarterly, which form of publication enables the Society to effect a considerable saving in the matter of postage. It is the belief of your Board that the most effective manner to increase the usefulness of the Society to its members is by the enlargement of its technical publications, and that, while the additional expense will be great, the financial condition of the Society warrants the expenditure, and it is hoped that members will do their part by furnishing original papers, and discussing those written by others.

MEETINGS.

During the year, 25 meetings have been held as follows: At the Annual Meeting, 2; at the Annual Convention, 5; and 18 other meetings held at the Society House.

At these meetings there were presented 17 formal papers, 8 of which were illustrated with lantern slides, besides which there were 4 topical discussions and 3 lectures, 2 of which were illustrated. There were also 7 papers published in *Proceedings* which were not presented at any meeting of the Society. The number of members and others who took part in the preparation of, or discussion of, these papers was 220.

The Fortieth Annual Convention was held at Denver, Colo., and the meetings, excursions and entertainments were well attended, instructive, and enjoyable.

The total attendance at the twenty-five meetings held was about 5 050. The registered attendance at the Annual Meeting was 653, and at the Annual Convention 438 (includes members only), but there were many members and guests present at each of these meetings who failed to register.

MEDALS AND PRIZES.

For the year ending with the month of July, 1907, prizes were awarded as follows:

The Norman Medal to Leonard M. Cox, M. Am. Soc. C. E., for his paper entitled "The Naval Floating Dock—Its Advantages, Design and Construction."

The Thomas Fitch Rowland Prize to James D. Schuyler, M. Am. Soc. C. E., for his paper entitled, "Recent Practice in Hydraulic-Fill Dam Construction."

FINANCES.

During the year the mortgage debt has been reduced by a payment of $10 000, and now amounts to $155 000.

The attention of members is invited to the Secretary's statement of receipts and disbursements, and to the general balance sheet which accompanies it, in which the very satisfactory financial condition of the Society appears.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

NEW YORK, JANUARY 5TH, 1909.

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen:—I have the honor to present a statement of Receipts 31st, 1908. I also append a general balance sheet showing the condition

Receipts.

Balance on hand, December 31st, 1907, in Bank, Trust Company, and in hands of Treasurer		$20 749.28
Entrance Fees	$11 869.55	
Current Dues	53 266.64	
Past Dues	1 570.40	
Advance Dues	23 407.40	
Compounding Dues	500.00	
Certificates of Membership	465.75	
Badges	2 654.88	
Sales of Publications	3 268.95	
Interest	610.15	
Library	496.54	
Convention	3.00	
Annual Meeting	1 272.50	
Binding	6 107.09	
Miscellaneous	139.65	
		105 632.50
		$126 381.78

YEAR ENDING DECEMBER 31st, 1908.

American Society of Civil Engineers.

and Disbursements for the fiscal year of the Society, ending December of the affairs of the Society.

Respectfully submitted,
Chas. Warren Hunt,
Secretary.

Disbursements.

Salaries of Officers	$11 533.31	
Clerical Help	14 436.86	
Caretaking	1 728.71	
Publications	24 313.03	
Library	824.18	
Library Maintenance	100.83	
Maintenance of House	177.08	
Heat, Light and Water	1 194.47	
Convention	1 309.80	
Annual Meeting	1 792.60	
Postage	6 857.26	
General Printing and Stationery	2 900.92	
Badges	2 246.50	
Certificates of Membership	337.60	
Prizes	120.00	
Binding	5 772.93	
Furniture	235.26	
Interest and Insurance	6 257.08	
Members' Accounts	48.80	
Current Business	957.53	
Petty Expenses	154.43	
Bond and Mortgage (Payment on Principal)	10 000.00	
		$93 299.18
Balance on hand, December 31st, 1908:		
In Union Trust Company	$18 470.69	
In Garfield National Bank	13 111.91	
In hands of Treasurer	1 500.00	
		33 082.60
		$126 381.78

GENERAL BALANCE SHEET, DECEMBER 31st, 1908.

Accompanying the Report of the Secretary.

Assets.

Three Lots (estimated value)		$270 000.00
Society Building (cost)		166 240.73
Furniture (cost)		18 165.82
Publications on hand (inventoried cost value)		19 745.14
Library:		
Cash expended for books, etc.	$13 030.44	
Donations, estimated value	53 687.84	
		66 718.28
Due from Members	$3 609.05	
Due from Non-Members	830.31	
		4 439.36
Cash		33 082.60
		$578 391.93

Liabilities.

Dues for 1909 paid in advance		$ 23 407.40
Mortgage Debt and Loan		155 000.00
Funds invested in Society House, Lots and Library	$ 26 730.78	
Surplus	373 253.75	
		399 984.53
		$578 391.93

We have examined the books and accounts of the American Society of Civil Engineers, for the year ended December 31, 1908, and certify that the foregoing Balance Sheet is in accordance therewith, and, in our opinion, correctly states the condition of the Society's affairs, as shown by the books.

79 Wall Street, New York.
January 14, 1909.

Marwick, Mitchell & Co.,
Chartered Accountants.

REPORT OF THE TREASURER.

In compliance with the provisions of the Constitution, I have the honor to present the following report for the year ending December 31st, 1908:

Balance on hand December 31st, 1907			$20 749.28
Receipts from current sources, January 1st to December 31st, 1908			105 632.50
Payment of Audited Vouchers for Current Business, January 1st to December 31st, 1908		$83 299.18	
Payment on principal of bond and mortgage		10 000.00	
Balance on hand December 31st, 1908:			
In Union Trust Company	$18 470.69		
In Garfield National Bank	13 111.91		
In hands of the Treasurer	1 500.00		
		33 082.60	
		$126 381.78	$126 381.78

Respectfully submitted,

Jos. M. Knap,
Treasurer, Am. Soc. C. E.

New York, January 5th, 1909.

ACCESSIONS TO THE LIBRARY

(From December 9th, 1908, to January 12th, 1909)

DONATIONS*

AIR LIQUIDE OXYGÈNE AZOTE.

Par Georges Claude. Préface de M. d'Arsonval. Paper, 10 x 6½ in., illus., 399 pp. Paris, H. Dunod et E. Pinat, 1909. 15 francs.

The author, it is stated, after an investigation of the subject, has become convinced of the necessity of a work on the liquefaction and separation of air into its elements such as is presented in this volume. The Contents are: Part I, La Liquéfaction des Gaz; Part II, La Liquéfaction Industrielle de l'Air; Part III, Conservation et Propriétés de l'Air Liquide; Part IV, La Séparation de l'Air en Ses Éléments.

RESERVOIRS FOR IRRIGATION, WATER-POWER AND DOMESTIC WATER-SUPPLY.

With an Account of Various Types of Dams and the Methods, Plans and Cost of Their Construction. By James Dix Schuyler, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 10 x 7 in., illus., 26 + 573 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $6.00.

The first edition of this book was issued in 1901; since then, it is stated, there has been such an increase in the number of dams and reservoirs constructed and such a demand for information regarding their dimensions, character, plans, materials, methods of construction, and cost, that the author feels justified in bringing out this, the second edition. It is stated that much new material has been added to the subject-matter, and that some which has become obsolete has been omitted. The chapter on Hydraulic-Fill Dams has been greatly enlarged by descriptions of later constructions of this type. Chapter III, on Masonry Dams, has also been enlarged, mention being made of the most notable dams in the world of this type, as well as of many of the smaller ones. Plates 1, 2 and 3, show profiles of all the better known masonry dams in the world, drawn to uniform scale for easy graphical comparison. Chapters have also been added to the sections on reinforced concrete and structural steel dams, giving the latest practice in these types. According to a secondary title, the book also contains miscellaneous data on the available water supply for irrigation in various sections of Arid America; distribution, application, and use of water; rainfall and run-off from various water-sheds; evaporation from reservoirs; effect of silt on the useful life of reservoirs; average cost of reservoirs per unit of capacity, etc. The Contents are: Rock-Fill Dams; Hydraulic-Fill Dams; Masonry Dams; Earthen Dams; Steel Dams; Reinforced Concrete Dams; Natural Reservoirs; Miscellaneous. There is an Appendix containing tabulated data of the cost of reservoir construction per acre-foot in the United States and in foreign countries on various types of dams, also tables of area and capacity of twelve Western reservoirs, at varying levels.

MORRISON'S SPRING TABLES.

A Handbook for Engineers, Students, and Draughtsmen. By Egbert R. Morrison. Cloth, 9 x 6 in., 84 pp. Sharon, Pa., E. R. Morrison, 1908.

In his book the author has divided springs into two classes, namely, light and heavy, called wire and bar in the case of helical springs and sheet and plate in the case of elliptical springs. The basis of calculation in helical springs is said to have been taken as the ratio between the diameter of the bar and the mean diameter of the springs; in elliptical springs, it is the ratio between the thickness of the plate and the span or net length of the spring. The author states that in the tables he has arranged the properties of light springs under graduated values of the fundamental ratio, and those of heavy springs under each size bar or plate. The working base, in the case of helical springs, is stated to be 1 in. of solid height, that for elliptical springs being taken as one plate 1 in. wide. All calcula-

*Unless otherwise specified, books in this list have been donated by the publisher.

tions are said to be based on a fiber strain of 80 000 lb. per sq. in. The modulus of elasticity for helical springs is taken at 12 600 000, and for elliptical springs at 25 400 000. The loads are given in pounds, and all dimensions are in inches. The Contents are: Part I, Formulas; Part II, Mathematical Tables Supplementary to Formulas; Part III, Spring Tables.

THE VENTILATION OF PUBLIC SEWERS.

By John S. Brodie. Cloth, 9 x 6 in., illus., 170 pp. London, The St. Bride's Press, Limited, 1908. $2.40.

The author states that he has considered the methods of sewer ventilation under three heads, namely, ventilation by natural air currents; ventilation by artificially produced air currents; and deodorization of foul sewer air or gas before it is discharged into the open air. Under each head he describes the methods used in various cities and towns in England, the laws governing their use, and gives abstracts of reports and papers by engineers and engineering societies on the subject. Chapter VI is devoted to a discussion of comparative costs of modern sewer systems. The Chapter headings are: Introductory; Is the Ventilation of Sewers Necessary?; Ventilation by Natural Air Currents; Sewer Ventilation by Artificially-Produced Air Currents; Deodorisation of Sewer Air or Sewer Gas; Comparative Costs of the Various Systems now in Use; Summary and Conclusions. There is an Appendix which contains abstracts of reports, papers, etc., on sewer ventilation, and an index of four pages.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES

For Buyers and Sellers. Especially Devoted to the Interests of the Architectural, Mechanical, Engineering, Contracting, Electrical, Railroad, Iron, Steel, Hardware, Mining, Mill, Quarrying, Exporting and Kindred Industries. Seventeenth Annual Edition. Cloth, 10 x 8 in., illus., 82 + 1240 pp. New York, Samuel E Hendricks Co., 1908. $10.00.

This is an annual index of the above-mentioned industries, and is said to contain more than 350 000 names and addresses of manufacturers and firms and upwards of 30 000 business classifications. Full lists of manufacturers of, and dealers in, everything used in the manufacture of materials, machinery, and apparatus for these industries are given. The contents are classified by subject, under which are given, in alphabetical order, and in some cases by States, the names and addresses of firms dealing in that particular article. There is an index of eighty-two pages.

LABORATORY NOTES ON INDUSTRIAL WATER ANALYSIS.

A Survey Course for Engineers. By Ellen H. Richards. Cloth, 9 x 6 in., 49 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. 50 cents.

In the Introduction it is stated that for manufacturing purposes there are special requirements to be considered under each case relating to the water to be used for such purpose, and that the engineer, in deciding upon recommendations, is often required to estimate the value of such water. In this short course on industrial water analysis only these special methods are considered, it is stated, the ordinary analytical processes being omitted entirely. Part I of the book consists of five laboratory exercises. In Part II, the Contents are: Standard Solutions; Computation of Hypothetical Combinations; Percentage Composition of Salinity in Various Waters; Tables; Convenient Data; Some Useful References. This last chapter contains a short bibliography of the subject.

THE MODERN ASPHALT PAVEMENT.

By Clifford Richardson, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 9 + 629 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $3.00.

The first edition of this work was published in 1905. Since that time there have been considerable developments in the industry, which have caused a com-

plete revision of the subject-matter in order to bring it up to date. The book includes, it is stated, descriptions of the forms of construction which have been found by engineers to be the most satisfactory; the character of the materials entering into the composition of asphalt pavements; the best methods of construction used at the present time, together with the reasons which led to their adoption; the proper methods of maintenance and the causes of their deterioration; and specifications for asphalt pavements to meet various uses. At the end of each chapter a brief résumé of the contents of that chapter is given. The Contents are: Part I, The Foundation and Intermediate Course; Part II, The Materials Constituting the Asphalt Surface Mixture; Part III, Native Bitumens in Use in the Paving Industry; Part IV, Technology of the Paving Industry; Part V, Handling of Binder and Surface Mixture on the Street; Part VI, The Physical Properties of Asphalt Surfaces; Part VII, Specifications for and Merits of Asphalt Pavements; Part VIII, Causes of the Defects in and the Deterioration of Asphalt Surfaces; Part IX, Control of Work.

A STUDY OF ORE DEPOSITS FOR THE PRACTICAL MINER.

With Descriptions of Ore Minerals, Rock Minerals, and Rocks. By J. P. Wallace. Cloth, $9\frac{1}{2}$ x $6\frac{1}{2}$ in., illus., 15 + 349 pp. New York and London, Hill Publishing Company, 1908. $3.00.

This book, it is stated, is a condensation of personal experiences covering a period of thirty years, the borrowed experience of practical mining men, and of facts and theories from all the available mining literature. It is said to be written for the miner, the prospector, and the mining public. A brief description of the most important minerals, ores, and rocks is given. The structural features of ore deposits and the walls enclosing them, together with the form, origin, and manner of occurrence of deposits, it is stated, have been given special attention. Descriptions of prominent mines of various types and forms, especially extensively developed properties with a history, are also given, together with a brief mention of the geology of each region. At the back of the book there is a list of the papers and publications which have been consulted. The Contents are: Part I, The Ore Minerals; Part II, the Rock Forming Minerals, Rocks and Rock Displacements; Part III General Characters and Classes of Ore Deposits; Part IV, Some Types of Ore Deposits; Part V, Mine Valuation and Prospecting.

THE STEAM TURBINE.

A Practical and Theoretical Treatise for Engineers and Designers, Including a Discussion of the Gas Turbine. By James Ambrose Moyer. Cloth, 9 x 6 in., illus., 7 + 370 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $4.00.

It has been the intention of the author to make this book a manual for the practical engineer for use in the design, operation, or manufacture of steam turbines, and to explain in a general way the more important problems necessary to the qualified steam engineer. He takes up first the more simple problems of nozzle design, which he follows with a discussion of blade design and the steam turbine generally. At the back of the book there is an entropy-total heat chart which is laid out with lines of constant superheat instead of lines of constant temperature, which are generally used for charts of this kind. The Contents are: Introduction, The Elementary Theory of Heat; Nozzle Design; Steam Turbine Types and Blade Design; Mechanical Losses in Turbines; Method for Correcting Steam Turbine Tests; Commercial Types of Turbines; Governing Steam Turbines; Low-Pressure (Exhaust) Turbines; Marine Turbines; Tests of Turbines; Steam Turbine Economics; Stresses in Rings, Drums, and Disks; Gas Turbines; Electric Generators for Turbines. There is an index of eight pages.

ELEVATOR SERVICE.

By Reginald Pelham Bolton, M. Am. Soc. C. E. Cloth, 11 x 8 in., illus., 69 pp. New York, Reginald Pelham Bolton, 527 Fifth Avenue, 1908. $5.00.

In a secondary title, it is stated that the book contains descriptions of operating conditions and proportions with diagrams, formulas, and tables for passenger travel, schedule and express operation, with the relation of the elevators to the building, and proportions and loads of cars. The Contents are: The Problem of Vertical Transportation; Operating Conditions; Passengers and Operators; Rating

the Work of the Elevator; Computing the Average Work; Express Service; The Shape and Size of the Car; Load and Speed Combinations; The Building and Its Proportionate Service; Examples of the Use of Figure XI; Definitions of Some Terms Used in Connection with Elevators.

STATICS BY ALGEBRAIC AND GRAPHIC METHODS.

Intended Primarily for Students of Engineering and Architecture. By Lewis Jerome Johnson, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 11 + 169 pp. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908. $2.00.

Several specific purposes have been attempted in this book, it is stated, to help students of engineering and architecture to acquire a knowledge of statics which will include the power to apply that knowledge correctly in practice. These purposes include: Attention to the starting points of science and clearness as to the course of deduction therefrom; the pointing out of the inherent mathematical limitations of pure statics and showing how all its important problems may be solved; the development of algebraic and graphic methods of solution side by side and with equal thoroughness; the presentation of a graded set of problems illustrating not only universal principles, but also how statics is used in engineering practice, etc. The most important additions to the subject-matter, in this edition, will be found, it is stated, mostly in the appendices, and consist of a detailed and time-tested scheme for assigning individual data for the exercises in the text and a collection of practice problems with answers, which have been collected from various sources. There are also a short syllabus and seven new double-paged plates which give solutions of certain of the exercises. The Contents are: Part I, General Principles and Methods: Introduction; Definitions and Preliminaries; Notation and Conventions; Parallelogram of Forces and Its Derivatives, etc.; Algebraic and Graphic Statements of the Conditions of Equilibrium with Applications; Scope of Pure Statics; Solutions of Statical Problems; Additional General Topics and Processes. Part II, Applications; Centers of Gravity; Stress; Structures; Stresses in Non-Framed Structures; Stresses in Framed Structures; Additional Topics and Examples; Appendices.

Gifts have also been received from the following:

Am. Ry. Assoc. 1 vol.
Am. Ry. Master Mechanics Assoc. 1 bound vol.
Am. Soc. Mech. Engrs. 1 bound vol., 1 vol.
Argollo, M. de T. e. 1 pam.
Belzner, Theodore. 1 vol.
Bezault, M. B. 3 pam.
Bissell, C. S. 1 pam.
Boston, Mass.-Eng. Dept. 3 pam.
Butler, M. J. 1 bound vol.
Cambridge, Mass.-City Engr. 6 pam.
Canada-Geol. Survey. 2 pam.
Canadian Min. Inst. 1 vol.
Chaumelin, Gaston. 1 vol.
Chemisches Laboratorium für Tonindustrie und Tonindustrie-Zeitung. 3 vol.
Columbia Univ. 1 vol.
Crane Co 1 pam.
Decimal Assoc. 2 pam.
East Indian Ry. Co. 1 pam.
Farley, G. P. 1 bound vol.
Fetherston, J. T. 2 pam.
Fuller, W. B. 1 bound vol.
Illinois-Bureau of Labor Statistics. 1 bound vol.
Illinois-State Highway Comm. 1 bound vol.
Indian Midland Ry. Co. 1 pam.
Indiana-State Board of Health. 1 bound vol.
Inst. of Civil Engrs. 1 bound vol.
Inst. of Engrs. and Shipbuilders in Scotland. 1 bound vol., 1 pam.
Inter. Waterways Comm. 3 pam.
Iowa-Board of R. R. Commrs. 1 bound vol.
Krug, Edmundo. 3 pam.
Lakes-to-the-Gulf Deep Waterway Assoc. 1 pam.
Lehigh & Hudson River Ry. Co. 1 pam.
Lewis Inst. 1 pam.
Lochridge, E. E. 2 pam.
London, Ont.-Water Commrs. 1 pam.
Manchester Steam Users' Assoc. 1 pam.
Massachusetts-State Board of Health. 1 bound vol.
Mass. Inst. of Tech. 1 vol.
Master Car Builders' Assoc. 1 bound vol.
Merchants Assoc. of New York. 2 pam.
Mexican Central Ry. Co., Ltd. 1 pam.
Michigan-Secy. of State. 1 bound vol.
Montreal, Que.-City Clerk. 1 bound vol.
Morley, Fred. 1 bound vol., 1 pam.
Nat. Board of Fire Underwriters. 15 pam.
New York City-Board of Estimate and Apportionment. 1 pam.
New York City.-Public Service Comm. 1 bound vol., 2 pam.
New York City-Registrar of Records. 2 bound vol.
New York City Record. 1 bound vol.
New York, New Haven & Hartford R. R. Co. 2 pam.
Ohio-Bureau of Inspection and Supervision of Public Offices. 1 pam.
Ohio State Univ. 1 pam.

Oklahoma-Geol. Survey. 1 pam.
Pennsylvania State College. 1 pam.
Philadelphia, Pa.-Mayor. 3 bound vol.
Platt, T. C. 1 bound vol., 3 pam.
Poughkeepsie, N. Y.-Board of Public Works. 1 pam.
Powell, F. W. 1 pam.
Providence, R. I.-Dept. of Public Works. 10 pam.
Queensland, Australia-Commr. for Rys. 1 vol.
Richmond, Fredericksburg & Potomac R. R. Co. 1 pam.
Richardson, Clifford. 1 pam.
San Francisco, Cal.-City Engr. 1 vol.
Siemens-Schuckert Werke, G. m. b. H. 1 pam.
Smithsonian Institution. 1 bound vol., 1 pam.
Stanton, R. B. 2 diagrams.
Towne, H. R. 1 pam.
Tracy, Patrick J. 1 bound vol.
Union Pacific R. R. Co. 1 pam.
U. S.-Bureau of Insular Affairs. 1 pam.
U. S.-Bureau of the Census. 2 pam.
U. S.-Bureau of Steam Eng. 1 pam.
U. S.-Chief of Engrs. 12 specif.
U. S.-Coast and Geodetic Survey. 14 charts.
U. S.-Commr. of Education. 1 bound vol.
U. S.-Forest Service. 1 pam.
U. S.-Gen. Land Office. 1 bound vol.
U. S.-Geol. Survey. 1 pam.
U. S.-Isthmian Canal Comm. 1 vol.
U. S.-Library of Congress. 1 bound vol.
U. S.-Office of Experiment Stations. 5 bound vol.
U. S.-Office of Public Roads. 2 pam.
U. S.-Office of the Library and Naval War Records. 8 pam., 1 vol.
U. S.-Weather Bureau. 1 pam.
Univ. of Minnesota. 1 pam.
Victoria, Australia-Sludge Abatement Board. 6 pam.
Wallace, John F. 1 pam.
Washington Southern Ry. Co. 1 pam.
Western Australia-Govt. Rys. 1 pam.
Worcester Polytechnic Inst. 1 pam.

BY PURCHASE

Methods of Treating and Disposing of Sewage. Fifth Report, with Appendices 1, 2, 5, 6, 7, and 8 of the Royal Commission on Sewage Disposal. London, Wyman & Sons, Limited, 1908.

Illustrated Technical Dictionary in Six Languages. Edited by Alfred Schlomann. Vol. 4, Internal Combustion Engines; comp. by Carl Schikore. New York, McGraw Publishing Co., 1908.

Handbuch der Ingenieur Wissenschaften. Edited by Th. Landsberg. Part II, Der Brückenbau. Vol. 1 and 2, Fourth Edition, and Vol. 3 and 4, Third Edition. Part III, Der Wasserbau. Vol. 3, 5 and 8, Fourth Edition. Leipzig, Wilhelm Engelmann, 1904, '06–07.

Elektrotechnik in Einzeldarstellung. Edited by Gustav Benischke. Vol. 13, Elektrotechnische Messungen und Messinstrumente. By Gustav Wernicke. Braunschweig, Friedrich Vieweg und Sohn, 1909.

SUMMARY OF ACCESSIONS

From December 9th, 1908 to January 12th, 1909

Donations (including 18 duplicates)	201
By purchase	16
Total	217

MEMBERSHIP

ADDITIONS

(December 9th, 1908, to January 12th, 1909)

MEMBERS

		Date of Membership.
BOOTH, GEORGE WILLIAM. Engr. in Chg. of Remodeling the Distribution System, N. Y. Dept. of Water Supply, 932 Park Row Bldg., New York City	Assoc. M. M.	Sept. 7, 1904 Nov. 2, 1908
BRYAN, FRED ASDEL. Gen. Mgr., Ind. & Mich. Elec. Co., 220 Colfax Ave., South Bend, Ind		Dec. 1, 1908
DOS SANTOS, JOSÉ AMERICO. Caixa 748, Rio de Janeiro, Brazil		Oct. 7, 1908
HIROI, ISAMI. Care, Mr. Miyoshi, 9 Thurleigh Rd., Wandsworth Common, London, S. W., England		Dec. 1, 1908
MCNAB, WILLIAM. Prin. Asst. Engr., Grand Trunk Ry. System, Montreal, Que., Canada		Dec. 1, 1908
WING, CHARLES BENJAMIN. Prof. of Structural Eng., Stanford Univ., 345 Lincoln Ave., Palo Alto, Cal	Assoc. M. M.	Nov. 4, 1896 Nov. 2, 1908

ASSOCIATE MEMBERS

BLACK, EDWARD FRYLING. Instr. in Civ. Eng., Anglo-Chinese Coll., Foo Chow, China	Jun. Assoc. M.	Oct. 31, 1905 Nov. 4, 1908
BURRAGE, JOHN OTIS. Asst. Engr. to City Engr., San Francisco, Cal	Jun. Assoc. M.	Mar. 1, 1904 Nov. 4, 1908
COOKE, SAINT GEORGE HENRY. St. Geo. H. Cooke Co., Cambridge Bldg., Chester (Res., 608 Morton Ave., Ridley Park), Pa		Jan. 5, 1909
DARROW, MARIUS SCHOONMAKER. Chf. Engr., Irrigated Lands Co., 427 D. F. Walker Bldg., Salt Lake City, Utah	Jun. Assoc. M.	Feb. 6, 1900 Dec. 1, 1908
DIEHL, DAVID LESLIE. Engr. and Contr. (Whittaker & Diehl) and (Ferro Concrete Co.), Union Trust Bldg., Harrisburg, Pa		Jan. 5, 1909
ENSIGN, GUERT WILLIAM. Asst. State Highway Commr., Harrisburg, Pa		Nov. 4, 1908
GENTNER, OTTO HENRY, JR. Engr. in Chg. of Estimating and Designing, Gen. Fireproofing Co., Youngstown, Ohio		Nov. 4, 1908
KLEINSCHMIDT, HENRY SCHWING. Constr. Engr., Utah State Land Board, Salina, Utah	Jun. Assoc. M.	Mar. 1, 1904 Dec. 1, 1908
LANE, EDWARD PERCY. Asst. Engr., N. Y. C. & H. R. R. R., 131 West 72d St., New York City		Jan. 5, 1909
LARSON, CLARENCE MELROSE. 913 University Ave., Madison, Wis		Oct. 7, 1908

ASSOCIATE MEMBERS (*Continued*).	Date of Membership.
McMORROW, JAMES WALTER. Secy., McMorrow Eng. & Const. Co., 360 West 125th St. (Res., 165 Audubon Ave.), New York City	Oct. 7, 1908
PERRY, FRANCIS WILLIAM. 117 Jerome St., Brooklyn, N. Y.	Jan. 5, 1909
ROUSSEAU, WILLIAM WHITE. Supt. of Constr., Bureau of Water Supply, 47 State St., Troy, N. Y.	Dec. 1, 1908
ROWLAND, WALTER. Draftsman. Isthmian Canal Comm., La Boca. Canal Zone, Panama	Dec. 1, 1908
SAMPLE, WILLIAM DWIGHT. Care, Madeira-Mamoré Ry., P. O. Box 304, Manaos, Brazil	Dec. 1, 1908
SKILLIN, EDWARD SIMEON. Second Vice-Pres., The Snare & Triest Co., 143 Liberty St., New York City	Dec. 1, 1908
TEIGEN, THOMAS WILLIAM ROSTAD. 192 West 5th St., St. Paul, Minn. { Jun. / Assoc. M.	Sept. 5, 1905 / Dec. 1, 1908
TURNER, AUGUSTUS MIESSE. 311 West 3d St., Greenville, Ohio	Dec. 1, 1908
WALTMAN, WILLIAM DEWITT. Supt., Porto Bello Rock Crushing Plants, Isthmian Canal Comm., Porto Bello, *via* Cristobal, Canal Zone, Panama	Dec. 1, 1908
WHEELER, ARTHUR CHAMBERS. U. S. Engr. Office, Honolulu, Hawaii	Oct. 7, 1908

ASSOCIATE

STROEBE, GEORGE GOTTLIEB. 315 New Eng. Bldg., Ann Arbor, Mich	Nov. 4, 1908

JUNIORS

ARMSTRONG, GEORGE SIMPSON, JR. 106 Penn St., Brooklyn, N. Y	Jan. 5, 1909
AYRES, LOUIS EVANS. With Gardner S. Williams, Cons. Engr., Ann Arbor, Mich	Dec. 1, 1908
CALHOUN, DAVID ADAMS. Chf. Engr., Richmond & Tide-Water Coal & R. R. Co., 29 Broadway, New York City	Oct. 6, 1908
CLEVELAND, LOU BAKER. Cleveland Bldg., Watertown, N. Y.	Sept. 1, 1908
EARL, AUSTIN WILLMOTT. 1050 Page St., San Francisco, Cal	Dec. 1, 1908
FEIGEL, JOHN HENRY. Asst. Engr., Buffalo Southern Ry., Care, Frederick K. Wing, 910 White Bldg., Buffalo, N. Y	Oct. 6, 1908
LAKE, ORLOFF. 2032 State Ave., New Orleans, La	Dec. 1, 1908
McCRORY, THOMAS GEORGE. Asst. Engr., International Contract Co., Seattle, Wash	Dec. 1, 1908
MARTINEZ, ROLANDO ARNOLDO. Sn. Miguel 168, Havana, Cuba	Dec. 1, 1908
ROBERTSON, AVALON GRAVES. Asst. Engr., Changuinola Ry., United Fruit Co., Bocas del Toro, Panama	Oct. 6, 1908

RESIGNATIONS

MEMBERS

	Date of Resignation.
MacRitchie, Charles	December 31, 1908
Whistler, Thomas Delano	December 31, 1908

ASSOCIATE MEMBERS

Harper, Austin Curtin	December 31, 1908
Schenk, Carl Louis Eduard	December 31, 1908

ASSOCIATE

Booth, William Ferris	December 31, 1908

JUNIORS

Fernow, Rossiter Raymond	December 31, 1908
Jordan, Willis Robert	December 31, 1908
Snyder, William Henry	December 31, 1908

DEATHS

Carothers, Daniel Dawson. Elected Member, April 4th, 1894; died January 1st, 1909.

DuBarry, Edmund Louis. Elected Associate, January 6th, 1875; died December 4th, 1908.

Green, Rutger Bleecker. Elected Junior, May 5th, 1896; Associate Member, October 5th, 1898; Member, September 6th, 1904; died December 8th, 1908.

McCurdy, John Egbert. Elected Member, April 1st, 1896; died December 15th, 1908.

Prince, Edward. Elected Junior, February 6th, 1878; Member, November 1st, 1882; died December 12th, 1908.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(December 8th, 1908, to January 11th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
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PAPERS AND DISCUSSIONS

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Vol. XXXV. JANUARY, 1909. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ACTION OF FROST ON CEMENT AND CEMENT MORTAR, TOGETHER WITH OTHER EXPERIMENTS ON THESE MATERIALS.

By Ernest R. Matthews, Esq.,* and James Watson, Esq.

To be Presented March 3d, 1909.

This paper describes in detail a series of experiments, extending over the past two years, made by the writers, in order to ascertain:

The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with—

(*a*) fresh water, cold or warm,

(*b*) sea-water;

The temperature below which it is detrimental to mix Portland cement concrete;

The effects produced by immersing concrete in—

(*a*) fresh water, hard or soft,

(*b*) sea-water;

Absorption of water by (dry) cement;

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Borough Engineer of Bridlington, England.

Quantity of water required to be added to cement to produce complete hardening.

The results of these experiments are not only interesting, but should prove of value to engineers generally.

The Hull Cold Storage Company, of Hull, England, kindly allowed the writers to use its refrigerating rooms in order to obtain the degrees of frost necessary for these experiments. The cement used was that manufactured by Robson's Cement Company, of Hull, and all the experiments were carried out at the laboratories of that firm, in Hull, these being kindly placed at the disposal of the writers.

Cement Used in Experiments.

The particulars of the cement used are as follows: Made on January 28th, 1907. Residues on a 5 776-sieve (that is, having 76 meshes per linear inch) = 0.5%; residues on a 10 000-sieve (having 100 meshes per linear inch) = 2.0%; residues on a 32 400-sieve (having 180 meshes per linear inch) = 11.5% (showing that the cement was ground extremely fine). Specific gravity = 3.112; flour = 54.5%; Le Châtelier tests, expansion = 2.7 mm.; Faija bath test, cement pat sound and hard; time of set of neat cement with 25% water: Initial set = 35 min.; permanent set = 6 hours, in a room kept at a temperature of 60° Fahr.

Tensile Strains. Neat Cement.

7 days	= 685 lb. per sq. in.	British Standard Test, 400 lb. per sq. in.
14 "	= 787 " " "	....
28 "	= 875 " " "	" " " 500 " " "

It will be seen that these results are well above the British Standard Tests.

Tensile Strains. One Part Cement and Three Parts Sand.

Mixed with three parts by measure of sand, and with no hammering of briquettes into moulds, its mean strength was:

7 days	= 200 lb. per sq. in.	British Standard Test, 120 lb. per sq. in.
14 "	= 277 " " "	....
28 "	= 333 " " "	" " " 225 " " "

Chemical Analysis.

	Percentage
Insoluble residue	0.82
Silica	20.43
Alumina	9.10
Oxide of iron	1.95
Lime	62.65
Magnesia	1.25
Sulphuric anhydride	1.38
Loss on ignition	1.66
Alkalies and loss	0.76
	100.00

The sea-water used in these experiments was taken from the North Sea; the fresh water was drawn from the Hull Corporation mains. To obtain the soft water, the temporary hardness of this water was removed, the permanent hardness being from 3 to 4 degrees.

The experiments herein will be compared with the tests just given, which will be referred to as the "Normal Tests."

Effects of Frost.

The effects of frost, and alternate frost and thaw, on the tensile strength of cement and cement mortar when mixed with (*a*) fresh water—cold or warm, (*b*) sea-water; and the temperature below which it is detrimental to mix Portland cement concrete, were determined by the following experiments:

Experiment A.—In this experiment the writers set out to discover the weakening effect, upon freshly mixed cement, of continuous light frost, temperature 29° Fahr., and of heavy frost, 15° Fahr. Nine briquettes were made with neat cement, 20% water, in the laboratory, the temperature of the air being 60° Fahr. These were taken from the moulds 24 hours after gauging and placed in cold stores, temperature 29° Fahr., and were broken at 7 and 28 days, respectively, the average tensile strength being:

At 7 days = 610 lb. per sq. in.
At 28 days = 905 " " " "

These results are compared with the normal tests as follows:

Tensile Strength, in Pounds per Square Inch.

	Normal tests, 24 hours in air at 60° Fahr., in water remainder of time.	In air at 60° Fahr. for 24 hours then 29° Fahr. for remainder of time.
7 days................	685	610
28 days................	875	905

In the 7 days' test it will be observed that there is a decrease of 10.9% in tensile strength, and an increase of 3.4% in the 28 days' test.

Experiment B.—Nine briquettes, made in the same manner as in Experiment *A*, were placed at 60° Fahr. Three were taken out at the end of 2 days, and placed in cold storage for 25 days; three more, at 7 days, for 21 days, and the other three, at 14 days, for 14 days. All were broken at 28 days, the result being as follows:

Tensile Strength, in Pounds per Square Inch.

	Normal test in air, 60° Fahr. for 24 hours, then in water.	In water at 60° Fahr. for 2 days, then in air at 29° Fahr.	In water at 60° Fahr. for 7 days, then in air at 29° Fahr.	In water at 60° Fahr. for 14 days, then in air at 29° Fahr.
28 days.......	875	912	977	942

Experiment C.—Nine briquettes, made as before, were allowed to harden in air for 7 and 28 days at 60° Fahr., the result being:

Average tensile strength at 7 days = 443 lb. per sq. in.
" " " " 14 " = 525 " " "
" " " " 28 " = 775 " " "

Days.	7, 14, and 28 days in air at 60° Fahr.	24 hours in air at 60° Fahr., then in water.
7	443	685
14	525	787
28	775	875

Experiment C_1.—

Sand and Cement Tests (3 to 1): Same as Experiments *A*, *B*, and *C*.

Days.	Normal test.	*A*.	*B*.	*C*.
7	200	163	....	220
14	277	270	....	250
28	353	342	*B*2. 317 *B*7. 305 *B*14. 243	322

Experiment D.—Effect of Alternate Frost and Thaw.—The briquettes were allowed to remain for 24 hours under damp flannel, then

in water for three days (60° Fahr.), then in water at the cold stores (temperature varying from 29° to 60° Fahr.). The briquettes were changed every three days.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
14	787	252	787	277
28	813	323	875	353

Experiment E.—This test was the same as *A* or *D*, but the briquettes were gauged with warm water; temperature, 100° Fahr.

Days.	Neat.	3 to 1.	*A* Test.	
			Neat.	3 to 1.
7	352	133	610	163
14	705	205	...	270
28	728	230	905	342

Experiment F.—(*Salt-Water Immersion*).—Nine briquettes were mixed with fresh water, and, after 24 hours, were immersed in sea-water, and broken at 7, 14, and 28 days.

Days.	Neat (20% water).	3 to 1 (10% water).	Normal.	
			Neat.	3 to 1.
7	770	242	685	200
14	742	278	787	277
28	812	360	875	353

Experiment G.—Nine briquettes were mixed with sea-water (same test as before).

Days.	Neat.	3 to 1.	Normal.		Mixed with Fresh Water and Immersed in Sea-Water.	
			Neat.	3 to 1.	Neat.	3 to 1.
7	693	180	685	200	770	242
14	775	287	787	277	742	278
28	773	293	875	353	812	360

Initial set, 9 min.; final set, 6 hours.

Experiment H.—Nine briquettes were mixed with sea-water, and, after 24 hours under damp flannel, were immersed in fresh water for the remainder of the time.

Days.	Neat.	3 to 1.	Normal.	
			Neat.	3 to 1.
7	628	150	685	200
14	733	255	787	277
28	713	297	875	353

Experiment K.—Same test as *A*, but the briquettes were kept in a temperature of 15° Fahr. in cold storage.

K.—Heavy Frost.			*A.*—Light Frost.		Normal.	
Days.	Neat.	3 to 1.	Neat.	3 to 1.	Neat.	3 to 1.
7	405	57	610	163	685	200
28	595	145	905	342	875	353

The briquettes were taken from the cold stores to the laboratory, two miles away, and were broken in a temperature of 60° Fahr., 45 min. after leaving the cold stores.

Experiment L.—The briquettes, 24 hours after gauging, were put into water at 60° Fahr. for 6 days, then placed in the cold stores at a temperature of 15° Fahr. for the remainder of the time.

Days.	Neat	3 to 1.	Briquettes placed in air at 60° Fahr. for a day or two after having been in cold stores at 15° Fahr. for 28 days.		*A.*	
			Neat.	3 to 1.	Neat.	3 to 1.
28	700	217	875	315	905	342

Experiment M.—The briquettes were put directly into the cold stores at 29° Fahr. for 7 days.

Days.	Neat.	Normal—Neat.
7	480	685
28	595	875

Experiment N.—The briquettes were made with neat cement, and placed, some in air at a temperature of 60° Fahr., and others in air at a temperature of 29.3° Fahr. (2.7° of frost). In 15 min. those in air at 60° Fahr. were still soft, while those subjected to frost had just frozen hard at the expiration of that time. Briquettes mixed with sand and cement (3 to 1) were subjected to a similar test. At the expiration of 15 min. those in a temperature of 60° Fahr. were still soft, while those in a temperature of 29.3° Fahr. had just frozen hard; and at a temperature of 27° Fahr. (5° below freezing point, Fahr.) were frozen very hard indeed at the expiration of that time.

Conclusions from the Foregoing Experiments.—These investigations have led the writers to the following conclusions:

(1) That light frost occurring 24 hours after the cement has been gauged, as indicated in Experiment *A* (3° of frost, or thereabouts), is detrimental to freshly mixed Portland cement, but only for a short time, and that at the end of 28 days it has quite regained its normal strength. If the frost occurs immediately after the cement has been gauged, the effect is more detrimental, and would appear to be permanent (see Experiment *M*). A minimum quantity of water should be added in frosty weather.

(2) That heavy frost (17° of frost, or thereabouts) has a most injurious effect (permanent) upon freshly mixed cement (neat), and cement mortar, as shown in Experiment *K*.

(3) That a light frost (3° of frost, or thereabouts), as indicated in Experiment *A*, does not affect cement or cement mortar if it has attained 2 days' set previous to the occurrence of the frost (Experiments *B*, *C*, and *D*).

(4) That the detrimental effect of light frost upon cement mortar (3 to 1) occurs more immediately than upon neat cement, but that cement mortar recovers from the ill effects of frost more rapidly than neat cement. At the end of 14 days it has quite recovered (Experiment *C*).

(5) That the mixing of cement or cement mortar with warm water (temperature, say, 100° Fahr.), which is sometimes done in frosty weather, and has been recommended by some engineers,* has a permanently injurious effect upon cement and cement mortar. This will be seen by reference to Experiment *E*.

* *Minutes of Proceedings*, Inst. C. E., London, Vol. CXXXIV, p. 384.

(6) That neat cement immersed in fresh water immediately after becoming set, and remaining in water, has a much greater tensile strength than when remaining continuously in air (Experiment *C*), the former being 55% stronger than the latter at the end of 7 days, 50% stronger at 14 days, and 11.3% stronger at 28 days. In 3 to 1 cement mortar, however, the result is different, the tensile strength being practically the same under both conditions.

(7) That it increases temporarily the tensile strength of cement and cement mortar to immerse them in sea-water 24 hours after gauging, instead of immersing them in fresh water; but the increased strength is only temporary (Experiment *F*).

(8) That there is an immediate reduction in the tensile strength of briquettes mixed with sea-water and immersed in sea-water (Experiment *G*) over those mixed with fresh water and immersed in sea-water (Experiment *F*). At the end of 14 days the strength of the former equals that of the latter, but after that period a depreciation again sets in, so that at 28 days there is a deficiency of 5% in the tensile strength.

(9) That the initial set of briquettes mixed with sea-water is 9 min.; permanent set, 6 hours. When mixed with fresh water, the former is 35 min.; the latter, 6 hours (Experiment *G*).

(10) That cement and cement mortar mixed with sea-water, and immersed in fresh water 24 hours afterward, have less tensile strength than when mixed with fresh water and immersed in fresh water (Experiment *H*).

(11) Experiment *L* shows that 17° of frost for 28 days does not kill the process of hardening in the briquette, but only delays it.

(12) It would appear from Experiment *N* that it is detrimental to concrete to mix it when the temperature is below 29.3° Fahr. (2.7° of frost), that being the freezing point of cement and concrete.

The Effects Produced by Immersing Concrete in Fresh Water (Hard or Soft), and in Sea-Water.

Neat Cement.—Neat-cement briquettes (mixed with 20% of fresh water) were immersed in fresh water (both hard and soft) and in sea-water after being in the moulds for 24 hours. In each case the absorption finished at the end of the seventh day: (1) The water absorbed by the briquettes in hard water at the end of the seventh

day = 3.46%; (2) by the briquettes in soft water = 2.92%; (3) by the briquettes in sea-water = 3.92 per cent.

Sand and Cement, 3 to 1.—As in the case of neat cement, just described, no absorption took place after the seventh day. (1) The briquettes in hard water absorbed at the end of the seventh day 3.4% of water; (2) in soft water, 3.4%; (3) in sea-water, 4.5 per cent. In the case of neat cement, and sand and cement briquettes, similar to the foregoing, but exposed to air for 7 days previous to immersion in water, the following results were obtained:

With neat cement during 7 days' exposure to air, the loss in weight was 1.7%; then, after 24 hours' immersion in (1) hard, (2) soft, and (3) sea-water, the increased absorption = 1.7%, just counterbalancing the previous loss. On continued immersion a further gain of only 0.3% in weight was obtained. The results of tensile tests at 28 days were: (1) = 875 lb.; (2) = 760 lb.; and (3) = 835 lb. per sq. in. The lower absorptions were most probably due to the partial set of the cement in the center of the briquettes during the 7 days' drying in the air, and the prevention of the free percolation of the water through it.

With sand and cement, during 7 days in air, the loss of moisture was 2.2%; then, after 24 hours' immersion, the gain was: (1) in hard water = 3.4%; (2) in soft water = 2.0%; and (3) in sea-water = 2.8 per cent. The results of tensile tests at 28 days were: (1) = 360 lb., (2) = 360 lb., and (3) = 330 lb. per sq. in.

Conclusions from the Foregoing Experiments.—(1) That no absorption takes place in neat cement, cement mortar, or concrete after immersion in water for 7 days.

(2) That neat cement immersed in hard fresh water for 7 days absorbs 0.54% more than when immersed in soft fresh water for the same period.

(3) That if immersed in sea-water for 7 days it absorbs 0.46% more than when immersed in hard fresh water, and 1.0% more than when immersed in soft fresh water.

(4) That cement mortar (3 to 1) absorbs the same quantity when immersed in either hard or soft fresh water, but when immersed in sea-water it absorbs 1.1% more.

(5) That neat cement and cement mortar, when immersed in hard fresh water, have practically the same absorption in both cases.

That neat cement immersed in soft fresh water absorbs 0.48% less than cement mortar (3 to 1), and that neat cement immersed in sea-water absorbs 0.58% less than cement mortar.

(6) That when neat cement is exposed to air for 7 days after gauging it loses 1.7% of its weight, but when immersed for 24 hours in water, either hard or soft fresh water, or sea-water, it absorbs 1.7%, which just counterbalances the previous loss.

(7) That after this loss has been made good, and the immersion of the briquettes is continued, there is a further gain of 0.3%, which averages 1.43% less than the absorption which takes place when cement is immersed in water 24 hours after gauging.

(8) That cement mortar (3 to 1) exposed to air for 7 days after gauging loses 2.2% of its weight; but after 24 hours' immersion in hard fresh water the absorption is increased to 3.4%; in soft water it is 2.0%, and in sea-water it is 2.8%, showing that soft water is absorbed more slowly than hard water, and that sea-water is also more slowly absorbed than hard fresh water.

Absorption of Water by (Dry) Cement.

Experiment 1.—Dry cement was compressed into moulds with a press, the same amount in each mould. The moulds were then allowed to stand in water for 24 hours. The result was an average increase in weight of 14.9 per cent.

Experiment 2.—The cement in the moulds was then changed so that there would be a variation in thickness of ⅛ in. The result was that the absorption varied correspondingly from 13.2 to 17.1 per cent. The writers then took a larger quantity of cement and compressed it into the same bulk as at first, and found that the absorption after 12 hours = 12.3%; after 24 hours = 14.4%; after 48 hours = 14.8%; after 4 days = 15.1%; after 8 days = 15.4%; after 31 days = 16.8% (constant).

Conclusions from the Foregoing Experiments.—(*a*) That when neat cement is immersed in water for 24 hours it absorbs from 14.4 to 14.9%, according to the thickness of the cement;

(*b*) That the absorption increases to 16.8% at the end of 31 days' immersion;

(*c*) That no absorption takes place after 31 days' immersion.

Quantity of Water Required To Be Added to Cement to Produce Complete Hardening.

According to the formula laid down by Le Châtelier,*

$$3CaO\ SiO_2 + \text{water} = CaO\ SiO_2\ 2\tfrac{1}{2}\ H_2O + 2Ca\ (OH)_2$$

35.4% of water would be required to be added to cement for complete hardening, and the hardened cement, therefore, would contain 26.2% of water in a combined state. From the figures the writers now give, this is found to be by no means the case. Possibly the reaction is not so complete, as Le Châtelier works it out by theory, and, although more water is combined by using an excess in gauging, yet the test becomes less and the strength is impaired, for Dr. Michaelis† found that, by sealing up some very finely ground cement with 150% of water for four weeks and then drying it over sulphuric acid, a loss on ignition was shown which equalled 27.5% of water absorbed. The results of the writers' experiments are as follows:

Experiment A.—Neat-cement test pieces gauged with 20% water, after 14 days' immersion in water, gave free moisture = 11.75%, combined = 9.7%; after 28 days' immersion they gave free moisture = 12.14%; combined = 10.9 per cent.

Experiment B.—Mortar briquettes, with 3 sand to 1 cement, gauged with 10% of water, after 24 hours were immersed in water and kept there for 7, 14, and 28 days:

	Free water at 105° cent. Percentage.	Combined water. Percentage.
At 7 days	9.44	9.20
" 14 "	8.49	10.58
" 28 "	8.68	10.29

Experiment C.—Neat cement test pieces were gauged with 20% of water, and, after 24 hours under damp flannel, were immersed in fresh water for periods up to 18 months.

	Free water at 105° cent. Percentage.	Combined water. Percentage.
After 3 days	8.0	6.4
" 7 "	8.9	7.9
" 28 "	9.0	12.38
" 3 months	9.25	13.05
" 6 "	9.55	12.95
" 12 "	9.40	12.90
" 18 "	9.82	12.20

* *Annales des Mines*, 1887.

† *Thonindustrie-Zeitung*, 1899.

Experiment D.—Other test pieces, made of different cement, were also tried, 20% of water being used for gauging, with the following results.

	Free water. Percentage.	Combined water. Percentage.
After 24 hours..................	8.5	3.3
" 48 "	7.45	11.5
" 72 "	7.10	7.7

Experiment E.—Frozen Test Pieces.—The cement was gauged with 20% of water, and, after 24 hours under damp flannel, was placed in cold storage at 15° Fahr. (17° of frost) for 6 days. The free water = 10.29%; combined water = 5.92 per cent. There was no active hardening of the cement.

Conclusions from the Foregoing Experiments.—That from the quantity of water absorbed by cement and cement concrete the writers are unable to determine precisely its ultimate effect upon the hardening of the concrete. It depends entirely upon physical conditions, such as the porosity of the concrete. When an excess of water is added, there seems to be a loss due to evaporation during the few hours after gauging; then, although immersed in water, the percentage of uncombined water remains constant, while the percentage of combined water increases until it attains a maximum of approximately 12 per cent. This occurs also with gauged cement exposed to open air, the only difference being that the quantity of free moisture gradually diminishes, owing to evaporation. In the case of sand and cement mortar, more water is absorbed in proportion to the quantity of cement used; this, most probably, is due to the greater porosity of the mass. The same result, it will be noted, occurs in the case of cement compressed into moulds, less water being absorbed in the same period when the mass is denser in bulk; so that, in the case of fire-proof floors and reinforced work, the denser the concrete, and with no more than the maximum quantity of water used to obtain a perfect gauging, the more fire-proof will be the mass, and less subject to dilatation through the expansion of unnecessary water. In the case of the test pieces subjected to hard frost, the process of hardening has been stayed, and, although the proportion of water used was the same as in Experiment *C*, yet in the same period of time 20% less

of the water had gone into combination, preventing the cement from attaining its normal strength.

The writers have endeavored to present what, in their opinion, is useful information concerning the behavior of Portland cement, cement mortar, and concrete, under various conditions. This information has been obtained by carrying out the series of experiments described herein, in which 727 briquettes were broken.

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PAPERS AND DISCUSSIONS

THE BONDING OF NEW TO OLD CONCRETE.

By E. P. Goodrich, M. Am. Soc. C. E.

To be Presented March 3d, 1909.

The necessity, in his general practice, of securing as perfect a bond as possible between masses of concrete which are deposited in abutting positions, but with longer or shorter intervals between the times of deposit, together with several demands for expert opinion as to the adequacy of the bond secured under certain given circumstances and with certain materials and methods, led the writer to make investigations into the history of the art, and to perform a series of experiments in the laboratory of the Polytechnic Institute of Brooklyn to determine how closely it is possible to approach a truly monolithic condition at such joints, when care is taken in executing the concreting in their vicinity.

Literature of the Subject.

The principal editorial in *The Engineering Record* of August 29th, 1908, discusses this subject with particular acumen. The remarks end with the statement:

"To sum up the whole matter in a phrase, the real obstacle militating against the bonding of fresh concrete to old, is dirt. This dirt may be an oily substance or a finely comminuted material forming an

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited, from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and when finally closed, the papers, with discussion in full, will be published in *Transactions*.

inert stratum constituting a most objectionable divided joint, or any similar substance which prevents the new crystals of setting from attaching themselves continuously to those already existing. It is commonly supposed and stated as a general proposition that new concrete cannot be bonded to old. There may be room for legitimate difference of opinion as to the practicability of attaining that end. Any process not feasible of execution under ordinary conditions of work may be so difficult as to be practically impossible, but some engineers of experience believe that bonding new concrete to old will not long be classed in that category. If such a bond be attempted with a truly clean surface experience shows that intelligent manipulation involving some pressure will insure invariably successful results. It has many times been done on a small scale, and it is not too much to expect that under suitable conditions further experience intelligently directed may accomplish the same ends under ordinary circumstances. Efficiency of concrete work will be greatly enhanced in many cases if such an important end can be uniformly attained; and when it can be done on a small scale, ultimate success on the large scale of usual engineering works is not too much to expect."

The use of oil in some form, coated upon the forms so as to prevent the adhesion of any concrete to them, is a direct cause of much trouble when a good bond is desired between the old concrete and any further concrete, stucco, grout, etc. The primary reasons for the use of this oil are:

(*a*) To prolong the life of the forms (the alternate wetting and drying tending to rapid deterioration), although the natural life is not usually long enough to make this item of much importance.

(*b*) To lessen the labor incident to the removal of the forms.

(*c*) To lessen the breakage of the forms due to sticking while being removed.

The handling of oil about any parts of the work is likely to provide sufficient material to add to the natural difficulty of securing good bond at far distant points. Men carry it on their clothes, their shoes, and their hands, and, as it usually floats on water, a thin film is likely to rise to the top surfaces of fresh concrete everywhere and remain on top of the concrete when the excess water evaporates. There is seldom enough alkali in the cement to saponify completely such oil within any appreciable lapse of time, and, even when such saponification does take place, the soaps thus formed are sometimes not readily soluble in water, and there is practically never sufficient water used to dissolve and remove them, even when they are soluble.

For these reasons, the writer has lately made it a practice not to coat his forms, or else to use soap for the purpose; either hard or soft soap is suitable, and the only drawback to its use is its relatively higher cost. Its prime advantage is its solubility in clear cold water. When soap is used, it is necessary merely to flush the surface thoroughly with water to remove the otherwise troublesome film. In securing a bond, the excess water is useful for other reasons, as well.

Laitance is often responsible for imperfect bond. This point is discussed in the following remarks by L. J. Hotchkiss* on "Some Details of Concrete Construction":

"Occasions will arise, however, where a longitudinal joint in a wall is necessary, and where this is the case the thorough cleaning of the concrete is often neglected. The laitance is not all removed and dirt, shavings and sawdust are permitted to remain in the forms. This work cannot be left entirely to laborers but must have efficient supervision and inspection. The surface of the concrete should be scrubbed with wire brushes and water until every stone shows clearly, and the water used in the scrubbing should be flushed off and not left standing in the forms with the laitance in suspension, to be deposited on the concrete again as soon as the scrubbing gang has gone. Both time and expense can be saved if the cleaning is done before the concrete becomes hard. It can be done with shovels and a small amount of washing instead of the protracted scrubbing necessary later. The bond obtained in even the most carefully cleaned joint is of uncertain strength, and if the cleaning is poorly done, there is no bond at all."

Shrinkage cracks almost invariably reveal themselves in reinforced concrete floors along the joints between successive days' work. After a time, retaining and other walls of mass concrete very often reveal even the horizontal joints between successive deposits, and the writer knows of one such wall through which water has found its way to such an extent along such joints as to ruin the appearance of the face of the wall and actually cause anxiety as to its continued stability.

Taylor and Thompson† cite an instance "In the New York Subway", where:

"Work was commenced with no provision for bonding horizontal layers, but it was soon found that more or less seepage occurred, and in one case where a large arch was torn down the division line between two days' work was distinctly seen."

* *The Engineering Record*, March 9th, 1907.

† "Concrete, Plain and Reinforced," p. 376.

In demolishing old concrete work with wedges and sledges, an experienced workman always looks for the joints between successive layers, and the writer has often seen large masses separated by using only a crowbar when the old seam had been discovered. The commonly observed hollow sounds which issue from cement sidewalks and floors under traffic when the floors and walks have been constructed with special cement-finish coats are always due to a separation having taken place between them and the base course. This is obviously due to poor bond. After any considerable lapse of time, stucco, mortar, plaster, and grout which have been applied to mass concrete work, are oftener found to have separated than to have retained a perfect bond with the backing.

Gillette and Hill* call attention to one primary reason why proper bond is difficult to secure on horizontal and other much-worked concrete surfaces, where the mortar has been flushed to the surface:

"Concrete which has set hard has a surface skin or glaze to which fresh concrete will not adhere strongly unless special effort is made to perfect the bond. * * * The secret of securing a good bond between fresh concrete and concrete that has set lies largely in getting rid of the glaze skin and the slime and dust which forms on it."

In a paper on "Construction Details of Reinforced Concrete Work",† De Forest H. Dixon, Assoc. M. Am. Soc. C. E., says:

"The sloping off of concrete in the beams and girders should never be allowed. It is a well-known fact that there is almost no bond between old and new concrete."

and again:

"Granolithic finish should be placed monolithic with the slab, or, if placed afterwards, which is desirable in stables, and some other classes of work, its thickness should never be less than 1½ in. Recently, an acid solution has been placed on the market which the author has tried in several instances, using, in some cases, finish as thin as ½ in., put on concrete that had attained an age of several months. This finish has been in place for three months, and thus far shows a perfect bond."

The specifications prepared for large engineering works to be executed in concrete usually contain a short clause with regard to bonding old and new concrete, but they are very rarely at all explicit,

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* "Concrete Construction, Methods and Costs."

† *Proceedings*, Municipal Engineers of the City of New York, 1907, p. 1.

the exact treatment usually being left to the resident engineer for determination. The following is the standard specification for work of this class, incorporated in all the published requirements for the several parts of the new Catskill Water Supply Development for New York City.

"For the proper bonding of new masonry to old, such provision shall be made of steps, dovetails, or other devices as may be prescribed. Whenever new masonry is joined to old, the surface of the old masonry shall be thoroughly cleaned, using stiff brushes and a stream of water; the laitance shall be scraped off and the surface shall be clean and wet at the moment the fresh concrete or mortar is placed. When ordered, a thick wash of rich mortar shall be brushed over the contact surface of the old concrete."

This specification is typical in the main, being rather better than most of the others in the writer's possession.

In the construction of tunnels, and heavy or extensive work of all similar kinds, joints are inevitable, and the designer must study each case carefully and devise proper quantities and arrangements for the reinforcement which is essential to securing the work against the appearance of shrinkage and temperature cracks along such natural joints. The subjects of setting, shrinkage, and change of size, from fluctuations of temperature and humidity, are intimately connected with the subject of bonding old and new concrete, but will not be discussed here, although they have all been investigated, both experimentally and analytically, by the writer in this and closely related work.

Another and a very vital and closely related question, and one which exists in the minds of most constructors, is as to the proper place in reinforced concrete floor-beam constructions at which to leave the joint between successive days' work. In the last analysis, if a perfect bond could be secured between the successively deposited masses, it would not matter in the least how or where one day's work were stopped. Usually, a compromise has been deemed best, and the joint made at about the quarter point in the span of a beam or slab, where neither the bending moment nor the shear is a maximum.

Patented Processes.

All the treatises on concrete discuss this important subject of bonding, but tell very little of real value. Ernest L. Ransome, Assoc.

Am. Soc. C. E., has secured several patents on the general treatment of concrete surfaces, with mechanical bonding devices and with acids, to secure a better connection; and other methods have been devised which make use of bituminous and other compounds to be applied to the old work and in preparing the new mortar. These are mostly used where it is the intention to apply stucco or plaster to mass concrete. Usually, these compounds are also claimed as resisters of dampness as well as bonding agents. Only one such process, not primarily for damp-resisting purpose, is known to the writer. In it a special preparation of coal-tar is used in the place of water in mixing the mortar for the new floor or sidewalk wearing surface.

The first patent (No. 647 904), issued to Mr. Ransome, concerning the subject of bonding, was dated 1898, and covers a "concrete joint strengthened with a metallic coil embedded in both of the joined masses."

Patent No. 694 578, also issued to Mr. Ransome, covers:

"The method of forming a bond between two bodies of concrete, which consists in first sprinkling over the surface of the first made body of concrete while yet soft and unset, a layer of prepared honeycomb slag, embedding the same into the surface, allowing the concrete to set and after removing all superfluous slag, covering it and embedding its exposed surfaces with the second body of concrete."

Patent No. 767 582, issued to Mr. H. L. Lewman, is

"In a concrete construction, adjacent sections connected by a bendable tie-piece having looped portions embedded in each section with its free ends meeting at its mid-length, and a connecting strip extending between said looped portions of the tie-pieces and having its free ends bent to intersect the body thereof, or with a rod extending through the opposite loops of each tie-piece at an angle thereto."

The best known of these patents is No. 800 942, issued to Mr. E. L. Ransome in 1905. It has to do with:

"The improvement in the art of forming structures of concrete and the like, which consists of forming a layer of plastic material containing cement, and permitting the same to harden, in removing portions of the hardened surface of said layer by the application of an acid thereto, in removing from said surface by washing the excess of acid and resultant salts and disintegrated material, and in forming a second layer of plastic material containing cement, over said surface of the first layer, and permitting the second layer to harden, whereby to form a perfect bond between the two layers."

In a pamphlet describing the use of a special mortar for cement floors, stucco, etc., Albert Moyer, Assoc. Am. Soc. C. E., gives the following method of securing proper adhesion between old and new cement in cement floor work:

"Sweep off the concrete surface thoroughly with a broom, wash the surface with clear water and sweep off with a broom. As soon as the excess water has been taken up by the atmosphere or absorbed in the concrete, paint the surface with a solution of one part commercial muriatic acid, three parts water; paint on three coats one right after the other. Within a few minutes wash off the surface with clear water, sweeping with a stiff broom. This removes all particles of dead cement which may remain on the surface, exposing each grain of sand and other aggregates which compose the concrete in very much the same condition as they were before being mixed. This also wets the concrete quite thoroughly.

"While the concrete is still wet, place an inch coat of mortar * * *."

The removal of efflorescence from concrete work, by the application of an acid wash, is so similar in method and result to that required in obtaining a perfect bond that the following description* may well be included for the sake of completeness:

Commercial hydrochloric [muriatic] acid, diluted with from 4 to 5 parts of water, was applied, and the surface scrubbed with common floor scrub brushes, water from a small hose being constantly played on the concrete while the cleaning progressed, so as to prevent the penetration of the acid. The cost on straight work was estimated at about 2 cents per sq. ft. cleaned. An experienced stone renovator should be employed whenever possible. Hydrochloric, acetic, and oxalic acids were experimented with, in conjunction with the scrubbing. All were effective. Hydrochloric acid was the best, requiring less scrubbing. Acetic acid came next in efficiency.

In this case the endeavor was simply to remove the thinnest possible layer from the surface, the acid being prevented from penetrating to any appreciable extent. In the patented process for securing decorative surface results on concrete by etching with acid, covered by letters patent No. 716 371 granted to Block and Richards, December 23d, 1902, the method is closely analogous to that necessary to secure a good bond, and is described as follows:

"The process of producing a natural finish on artificial stone composed of cement and particles of natural stone by subjecting it when

* Abstracted from *Engineering News*, October 31st, 1901.

molded and dried to a solution of muriatic acid, then rinsing the same in water, then subjecting the same to a solution of carbonate of soda, and finally rinsing the same in water."

Published Experiments.

Few experiments have thus far been published with regard to methods of bonding masses of concrete, but in *Engineering News* of August 13th, 1908, is given an abstract of a graduation thesis prepared by Raymond B. Perry, of Case School of Applied Science. He prepared and tested by flexure small concrete-beam specimens, $2\frac{3}{4}$ in. square and $27\frac{3}{4}$ in. long, which had been spliced at their centers. A variety of methods of splicing were used, none of which developed the strength of an unspliced specimen. His results and deductions are repeated here for future reference and comparison.

TABLE 1.—Perry's Tests.

Condition of Joint Surface.	Roughened.	Smooth.	Rough.	Smooth.	Moulded Groove.	No Joint.
Treatment.	Wet.	Grouted.	Grouted.	Ransomite.		
Results of tests, in pounds per square inch	158 123 87 124 128	119 131 207 82 124	279 211 237 225 227	232 128 220 231 146	173 133 120 128 110	255 230 249 271 257
Averages, in pounds per square inch...	124	133	236	191	123	252

"While experiments of so limited character are not absolute in their results, it may be concluded with some degree of certainty:

"(1) That the bond existing between new mortar or concrete and old, where the old surface is smooth, is very slight.

"(2) That about one-half of the strength of the concrete is developed in a joint bonded (*a*) by roughening the old surface; (*b*) by applying a layer of cement paste; (*c*) by providing the old surface with a bonding groove.

"(3) That a large part of the strength of the concrete, perhaps as much as 90%, is developed where the old surface is roughened and a layer of cement paste is applied.

"(4) That such a solution as 'Ransomite' practically takes the place of the roughening, since a bond made with it is otherwise similar to the one made in these tests by roughening the old surface and applying a layer of cement paste."

"Ransomite" is a powder which is to be dissolved in water in order to obtain the bonding solution.

These tests closely followed some experiments described in *Annales des Ponts et Chaussées*, Pt. 3, Vol. 27, 1907, the results of which were summarized as follows in *Engineering News* of December 12th, 1907:

"A [neat] cement joint is an aid in connecting old concrete work to new, and * * * whether this joint is used or not, an excessive tamping of the new work near the joint will add appreciably to the strength of the connection."

TABLE 2.—FRENCH TESTS.

SPLICED AFTER:	7 DAYS.		30 DAYS.	
Treatment of joint.	No grout.	Grouted.	No grout.	Grouted.
Results of tests, in pounds per square inch	24 97 67	209 175 300	171 154 142	159 302 227
Average, in pounds per square inch.........	63	223	156	229

It is stated that the above splices, made after 30 days, were better tamped than the earlier ones. It is also to be noted that the mechanical arrangments of the test brought a considerable direct compressive stress upon each joint, and in consequence these tests are not strictly comparable with any others.

The specimens were of the same size and kind as those described above, and all are open to the objection of revealing results comparable only with one another, as there is at best only an approximate relation between the maximum stresses developed in bending and those produced under direct stress.

Professor Morsch* gives the following results, showing that the direct stress may be considered to be about one-half that developed in bending when computed according to the usual formula. The stresses are in kilograms per square centimeter.

TABLE 3.—MORSCH'S TESTS.

Mixture.	1 : 3		1 : 4		1 : 7	
Percentage of water............	8	12	8	14	8	14
Bending stress............	21.4	23.2	16.1	16.7	13.3	12.8
Direct tension......	12.6	10.5	9.2	8.8	4.4	5.5

* "Eisenbetonbau," pp. 35-36.

The writer is not aware of any experiments having been made with regard to the bonding qualities of stucco or cement finish for concrete floors or sidewalks, except a single test with brick as a backing, made by Professor Ira H. Woolson, at Columbia University, for the exploiters of a special water-proofing cement mortar. His report reads in part as follows:

"Description of Specimens.—Two specimens were submitted for test, and each consisted of a common red building brick, with a coating on one side of about one-half inch of the cement. The cement was thoroughly dry and hard.

"Object of the Test.—The object of the test was to ascertain what degree of heat the cement would sustain without injury, either by cracking or by disintegration.

"Method of Test.—In order to develop the full effect of possible differences in coefficients of expansion due to heat in the two materials, the specimens were heated on the cement side only, and the brick kept cool as it would naturally be in a wall, where the face of the wall was heated. To accomplish this object, the specimens were laid, one at a time, horizontally before a side opening in a small gas blast furnace. To insure that the heat be uniformly distributed over the whole face of the cement, a $\frac{3}{16}$-inch sheet of composition asbestos board was used to close the furnace opening, and the cement face was placed near to it, but not touching.

"In this narrow air space between the cement and board the thermocouple of a Le Châtelier electrical pyrometer was placed, by which the temperature of the face of the cement would be accurately and continuously recorded.

"The heat was raised gradually and temperature read every three minutes. The total time of heating the first specimen was forty-five minutes, and the temperature attained was 1 000 degrees, F. At the time no cracks or other defects were noticeable in the cement, and the brick was cool enough to allow its being easily removed from the furnace by the bare hand. However, in a few minutes after removal, a crack developed along the line of juncture, between the cement and brick, which gradually spread until it had separated about half of the cement from the brick.

"The cement adhered to the brick so firmly that the crack in places ran into the body of the brick itself. No other disintegration occurred.

"The second specimen was heated the same way, but the time of heating was reduced to fourteen minutes, and the temperature to 600 degrees, F.

"This specimen showed no effect whatever due to the heat either hot, or later when it had fully cooled,

"It is quite evident that the cracking of the first specimen was due to unequal expansion of the brick and cement, and rapid cooling. It is possible that the cracking might not have occurred if the specimen had cooled slowly."

While the engineering periodicals have published quite a number of communications from time to time with regard to the proper points in concrete floor construction at which to stop work, still these discussions have all been of an entirely academic nature, no experiments having been made with regard to this fact, so far as known to the writer, except in the case of the reinforced concrete reservoirs for Mexico City, described by James D. Schuyler, M. Am. Soc. C. E.*

There are four reservoirs, all circular in form, and 105 m. in diameter at the top. The roof consists of a 6-in. slab of reinforced concrete supported by radial and concentric beams, resting on cylindrical pillars of concrete, all properly reinforced.

"In the building of such a massive roof over so large an area as each of these reservoirs presents, the question of sustaining the forms during the time required for the setting of the concrete was one of the most serious problems to be solved. To build up timber supports of requisite stiffness and stability reaching from the floor to the roof, on account of the height, would have required a vast forest of timber, of little subsequent utility, and costly to erect and remove. It was finally determined to suspend the forms from the tops of the columns by a series of steel I-beams of standard sizes and lengths which would have a market value for building construction of at least 75 per cent. of their cost after the works were completed. But in order to place the top series of I-beams high enough to act as suspensors for the forms of the main reinforced concrete beams, it was necessary to build up the capitals of the columns to the ultimate height of the top of the roof as bases on which to rest the temporary hanger beams, and in these concrete capitals, which form the ends of four intersecting concrete beams, mold all the steel reinforcing rods subsequently required for the concrete beams. This in effect meant the formation of the ends of the beams a long time prior to their ultimate completion. Hence to determine the effect of this composite construction of a beam, with a joint near each end, upon the final strength of the beam, a series of tests were made in the Condesa testing laboratory, with four full-size beams reinforced as they are to be used in the roof of the reservoir. Two sets of tests were made, with two beams each, 14 and 28 days after molding. These beams were 14 ft. 9 in. long, $19\frac{5}{8}$ in. deep, $7\frac{3}{8}$ in. wide, with a clear span of 13 ft. 1 in. between supports. In

* *The Engineering Record*, March 28th, 1908.

one beam of each set, the two ends were molded two days in advance of the remainder, in the form shown by the shaded areas of the diagram, leaving a well-defined line of jointure between the concrete of the ends and the central portion of the beam.

"Test A, at 14 days, was made under the following conditions: The solid beam had been molded complete at one operation, with 1:2:3 concrete, reinforced at bottom with two square corrugated bars ½ in. by 14 ft. 10 in.; two diagonals ½ in. by 15 ft. 6 in.; three oblique bars ½ in. by 5 ft. 2 in. at each end, and diagonals, from center to ends, ½ in. by 7 ft. 6 in., the metal in all being 0.7 per cent. of the beam. With 3 809 kilos applied at two points at one-half meter from central axis, the deflection was 2.4 mm.; with 6 185 kilos applied load, the deflection was 4.6 mm.; with 10 369 kilos the deflection was 8.2 mm., when three cracks appeared, half the depth of the beam, at distances of 1.5, 2.9 and 3 meters from one end. The final rupture occurred under a weight of 11 460 kilos.

"The second beam of the pair, molded in sections as described, showed a deflection of 2.4 mm. under 3 908 kilos, 4.8 mm. under 6 815 kilos and 8.0 mm. under 10 369 kilos, when four cracks appeared, 20 cm. long, at distances of 1.5, 1.7, 3.0 and 3.1 meters from one end, all inclining toward the load. The final rupture took place with a deflection of 20 mm. under a load of 13 648 kilos without the slightest sign of weakness appearing at the joints in the concrete. * * *

"Test B, at 28 days, gave results quite in parallel with those of Test A. The mixture was the same, but the reinforcing metal was greater, amounting to 1.0 per cent. of the beam, consisting of three straight base bars in bottom, two of which were ½ in. square and one ⅝ in. square, 14 ft. 10 in. long; also two base bars with diagonal to top at ends, each ½ in. square and 15 ft. 6 in. long; three intermediate top and diagonal bars at each end, each ½ in. square and 5 ft. 2 in. long and two central diagonal and top bars, each ½ in. square and 7 ft. 6 in. long.

"The beams showed the following comparative results during test:

" Applied Load. Kilos.	Solid Beam. Deflection.	Beam Molded in Sections. Deflection.
3,809	2.0 mm.	1.8 mm.
6,815	3.0 mm.	3.2 mm.
10,369	7.0 mm.	5.6 mm.
12,556	10.0 mm.	7.2 mm.
14,742	13.0 mm.	8.8 mm.
16,929	22.0 mm.	10.2 mm.
10,018	Ruptured by compression.	
19,116	....	12.4 mm.
21,299	....	16.4 mm.
22,390	....	Ruptured by compression.

"In each beam, cracks began to appear under a load of 10 369 kilos. * * *

"In this case, also, the absence of any cracks or sign of weakness on the line of joints between the older and newer portion of the concrete beam seemed to prove conclusively that there was no weakness to be feared from this fragmentary method of molding as compared with the monolithic beams, but, on the contrary, they developed greater strength in each case."

The Writer's Experiments.

It did not seem necessary to experiment as to the necessity of a thorough cleaning of the old concrete surfaces. This requirement is so obvious that it is surprising to note the way many other constructors apparently ignore it. A number of years ago, the writer discovered the advantages of using a jet of steam for this purpose, and in *The Engineering Record* of March 7th, 1907, he advocated its use, and gave several other hints on concrete work. The only other engineer who has advocated it is Mr. Ernest McCullough, who writes as follows:*

"Nothing is so absolutely bad for joints as sawdust and nothing is so hard to get rid of. Shavings and blocks of wood are picked up with rag pickers' sticks, which are pieces of wood about one inch square having a sharpened nail driven into one end. Loose gravel, etc., can often be brushed out. Sawdust however remains. Even a strong stream of water fails to get rid of it. Live steam at a high pressure will however clean off the surface of the concrete to the bone. It removes all the half-set and dead cement and all the sawdust.

The writer's experiments were of three varieties, and were arranged with the following objects in view:

(*a*) Measurement of actual tensile strength with five methods of bonding, applied to two varieties of surfaces, and after two different time intervals;

(*b*) Determination of the qualitative effect of exaggerated temperature changes on samples representing cement floor or sidewalk finish, placed after a relatively long lapse of time, with the help of acid treatment, as compared with simultaneous placing of the bottom and top layers;

(*c*) Determination of the advisability of depositing concrete for a deep beam in separate layers with intervening time intervals;

(*d*) Determination of the best point at which to splice concrete floor members between successive days' work;

* "Reinforced Concrete," p. 108.

(*e*) Value of acid treatment in ordinary floor and walk work.

The tension tests were made on standard cement briquettes. Only 1:2 (by weight) Portland cement mortar was tested.

All specimens were cured in air only, to be more nearly analogous to practical working conditions. Splices were made after intervals of 24 hours and of 7 days. All specimens were broken at the end of 30 days. For the sake of comparison, unspliced specimens were moulded and broken after the same period. One series of briquettes was made and broken after 24 hours. The half briquettes thus obtained were utilized in one series of experiments, half of them being spliced after a lapse of 24 hours and the other half after 7 days. It was considered that this series would represent conditions when the surface of old work was entirely broken away and all loose material removed just before the new material was deposited. Another series of briquettes was moulded with a thin strip inserted at the smallest section. When removed from the moulds, of course, these briquettes fell apart into two pieces, the surface next the partition having the exact appearance of having been moulded against a common wooden form. After 24 hours, and 7 days, these half briquettes were spliced, the surfaces to be bonded first being indented with a sharp steel tool in just such a form as would be given with a pickaxe or blunt chisel, according to the usual methods of roughening surfaces to be joined. Five methods were used in making bonds, or rather in connection with the making of the bond:

(*a*) The soft mortar was moulded against the half briquette with only as much manipulation as is usual in depositing concrete work. The old half was used dry in its natural state.

(*b*) The old half briquettes were first soaked in water for about 10 min. and then the new end was moulded as in (*a*).

(*c*) A creamy mixture of neat cement was prepared, and the ends of the half briquettes were dipped in it. This formed a coating similar to that most often specified and used. The new ends were then moulded in the usual way, before the grout had set to any great extent.

(*d*) The ends of the old briquettes were liberally washed with a 1:10 solution of commercial hydrochloric acid. They were then washed in clear water to remove all unspent acid, and the new ends were moulded as usual.

(*e*) A solution of "Bonsit" (a powder similar in nature to "Ransomite") was prepared in accordance with the directions sent with the can, or at the rate of 5 lb. of the powder to 10 gal. of water. This solution was then used in the same manner as in (*d*).

Three specimens representing each age, type of surface, and method of bonding were prepared.

Mortar specimens were decided upon, because it was believed that it is the mortar which has been flushed to the surface and against the forms on each side of the joint, which is the real bonding medium.

Briquettes were used for several reasons:

(*a*) Measurements of the true tensile strength of the bond could be most easily obtained.

(*b*) With the small size of the specimen it was thought that one would be more likely to obtain more uniform results than with specimens of larger size.

(*c*) If good results could be obtained with small specimens, a standard would be provided to be applied over larger areas.

There may be some question as to the truth of Assumption (*b*), because, over larger areas, local deficiencies would be merged with strong sections, so that a somewhat smaller variation would probably be obtained. The actual wide variation, in some cases, is instructive as to the probable conditions at different points in any joint. The narrower range found with some methods of bonding demonstrates, if anything at all, that it is possible with them to gain a better distribution of stress over larger areas than with the other methods.

The powder when put into solution gave a white precipitate when tested with barium chloride, showing that the acid in this case is sulphuric.

The results obtained from the tension bonding tests are contained in Table 4.

While the results in Table 4 are not as concordant as might be wished, still certain deductions may safely be made:

(*a*) The best results were obtained with surfaces having the roughness produced by the natural fracture. This points toward a purely mechanical bond, or one with little cementitious action.

(*b*) Practically no bond was obtained on surfaces which were moulded against forms and had dried for 7 days. This is shown by the large percentage prematurely broken, even with careful handling,

and the relatively low results obtained at best. Thorough soaking of the old work is essential.

TABLE 4.—GOODRICH BONDING TESTS.

	Rough Surface.				Smooth Surface.			
	24 Hours.		7 Days.		24 Hours.		7 Days.	
	Each.	Average.	Each.	Average.	Each.	Average.	Each.	Average.
Plain......	161 *C* 92	126	285 313 343	314	162 205 80	149	*C* *H* 15	15
Soaked....	*H* *H* 60	60	138 218 212	189	165 90 208	154	*H* *C* 44	44
Grouted...	380 315 *C*	347	347 280 200	276	98 43 160	100	*H* *H* *H*	......
HCl	68 *C* 155	112	126 175 221	174	87 232 105	141	83 51 44	53
"Bonsit"..	30 60 *H*	45	152 105 95	117	79 54 68	67	172 26 114	104
Average...	147	......	207	......	122	......	69	......

Whole Briquettes.
363
384
309
438
415
304
427
Av. 377

H = Broken while handling before expiration of 30 days.
C = Broke in testing machine while adjusting clips.

(*c*) It was noted with regard to the 7-day, smooth surface, grouted bond, that the water was absorbed so rapidly from the grout as to render it an actual detriment. However, the results obtained with regard to the 24-hour and 7-day, rough surface, soaked, specimens might lead one to infer that there is a possibility of using too much water. This latter may be the case in laboratory work, but, in the writer's building experience, few laborers have ever been met who were prone to use water to what seemed an excess. In applying stucco work, or lining reservoirs, however, an expert workman knows that there is a real possibility of too completely saturating the old work,

and it requires long experience and great care to obtain exactly the proper humidity.

The use of dry cement sprinkled over old concrete surfaces which have first been dampened is of questionable value, in view of the writer's experience.

(*d*) A coating of grout is advantageous, first having soaked the old concrete thoroughly to prevent the water in the grout from being absorbed too rapidly.

(*e*) Acid treatment at an age of 24 hours is not of appreciable value.

(*f*) On surfaces originally moulded against forms, after having dried for 7 days, acid is advantageous.

(*g*) Results with the use of acid were more consistent among themselves than with any other method.

(*h*) Why the splices made after the 7-day interval are so much better than those made after 24 hours on the rough surfaces, is difficult to explain, unless it be that the breaking to obtain this rough surface had loosened many small particles which still clung to the damp surface while only 24 hours old, but which were naturally removed during the handling incident to the 7-day tests.

(*i*) The bonds secured on rough and smooth surfaces after 24-hour intervals are not materially different.

While the acid treatment does not give a bond as strong as when the old surface is rough-pointed, still its cost is much less, and where an absolutely monolithic condition is not essential, but where fairly good bond is required, the use of acid is of value.

Again, where proper reinforcement is provided across joints, the use of acid is advisable and sufficient.

The powdered material is much more convenient to handle than the commercial acids shipped in glass carboys.

In view of the writer's and the other bonding experiments quoted, the following may be given as a proper mode of procedure:

"For connections made after a lapse of 24 hours or more, break back the surface concrete to firm material, and clean the fresh surface with steam, air blast or forceful water streams so as to remove all fine loose material. Saturate well, but not so that water stands on the surface or oozes from the material. Paint completely with neat cement grout, mixed to the consistency of thin cream, just before new concrete is deposited, and see that the latter is of proper mixture, containing a proper proportion of mortar, which should be worked against the joint so as to be certain that no voids exist in its vicinity.

"For connections made after long intervals, so that the old cement has set hard, and where the expense of rough-pointing the whole surface is greater than is required because of the nature of the desired bond, use commercial muriatic acid, diluted with clear water, 1 to 5, or the commercial bonding powders, dissolved in clear water at the rate of 5 lb. of powder to 10 gal. of water. First wet the old concrete surface with so much water that a fresh wetting is not immediately absorbed. Remove any excess of moisture and, when the surface appears as if commencing to dry, paint on the old surface three successive coats of acid one after the other. Let this remain for about 30 min., after which carefully clean the surface of unspent acid, soluble salts, and fine material, with plenty of water, finally cleaning with a steam jet or air blast if obtainable. Just before the fresh concrete is to be deposited, and while the old material is still very damp, apply a thin coat of neat cement grout mixed to the consistency of thin cream, just before the new concrete is deposited, and see that the latter is of proper mixture containing a proper proportion of mortar, which should be worked against the joint so as to be certain no voids exist in its vicinity."

A block representing a small section of a concrete sidewalk, or floor, was also prepared. It was just of a size to fit a furnace door under a high-pressure steam boiler. The concrete base was moulded 3 in. thick, in two operations, each covering half the area of the block. Reinforcing rods were left protruding from the first part deposited, so as to cement it firmly to the second one. The latter was placed 7 days after the first one. The surface of the latter was prepared to receive the finish coat by applying a solution of "Bonsit," according to the specification previously given. As soon as the concrete of the second section had partially set a ½-in. coat of 1 : 3 cement mortar was spread over the whole area of the block and well troweled so as to secure as good a bond with the backing as possible. The whole was allowed to set for 30 days, and was then subjected to an alternate heating and cooling by being placed first in the furnace door for a few minutes and then allowed to cool in the outdoor air. These alternate periods of heating and cooling were as follows:

Heated 12 min.	Heated 7 min.
Cooled 13 "	Cooled 15 "
Heated 15 "	Heated 5 "
Cooled 35 "	Cooled 15 "
Heated 5 "	Heated 10 "
Cooled 13 "	Cooled.

The temperature on the mortar face (the one toward the fire) was allowed to rise until steam was seen to issue from the edge of the block between the mortar and the concrete. The temperature at the depth of the joint was then approximately 200° Fahr. The exposed face was much higher, while what would be the bottom of the floor or sidewalk was not more than 60° Fahr. when tested by the hand. The block was then removed from the fire and allowed to cool until the hand could rest comfortably upon what had been the exposed face. This would correspond to a temperature of about 120° Fahr. The heating operation was again repeated, the time of exposure again reaching a point when steam appeared. After the second heating, the back of the block reached a temperature which was just a little too hot for the hand. At each cooling this fell to about 120° Fahr., but at the same time the mortar face must have had a range of temperature of at least 100 degrees.

After the second heating, a minute examination of the edges disclosed a hair crack along the joint between the two layers of the oldest and the newest materials; when the block had cooled so that it was about ready for another heating, a fine crack was also observed across the top along the line between the two parts of the backing. Evidently, a separation would have taken place except for the reinforcing bars. This crack could not be found on the bottom of the block, and it closed up entirely during each heating, appearing again during each successive cooling.

After the third heating, a narrow strip around two edges sounded hollow when tapped with a hammer. After each successive heating and cooling, this hollow area increased somewhat, until, after the fifth heating, it included about half the area over the older section. After the final heating, however, when a hammer was used to break away the loosened surface coat, not more than two-thirds could be thus loosened, some parts carrying off adjacent portions of the backing. That portion in which the top and bottom were placed at the same time could not be separated with a clear plane of cleavage, even after the specimen was broken into fragments. In the latter condition the two parts of the other half of the block readily parted under a smart blow. However, from the writer's experience, and from a study of the conditions of the test at the site during its progress, it can be positively affirmed that the value of the acid treatment was demonstrated by the adhesion still remaining even after the repeated heat applications.

No physical or weather conditions are ever as severe as those provided in connection with the furnace, and, even if each heating and cooling might be likened to an annual experience, ample bond remained after five summers and winters to warrant a contractor's guaranty, if the whole bonding operation were faithfully executed. Under actual weather conditions, the changes of temperature are not nearly as sudden or widely varying, so that the effects of the slightly different coefficients of expansion are very small.

On the basis of coefficients of 0.00000655 and 0.00000795 for mortar and concrete, as given by Bonniceau in *Annales des Ponts et Chaussées,* and with a temperature difference of 100° Fahr., there would be a difference in length of only 0.00000140 for each degree, or 0.000140, so that in blocks 100 in. square, in which the maximum probable action would be more than 50 in., the difference would be only 0.007 in. On the assumption of a modulus of elasticity of 2 500 000, the resulting tensile or compressive stress (ignoring any reduction in amount due to resulting strains) would be only 3½ lb. per sq. in. if uniformly distributed, or at that rate if concentrated at any one or more points. Whenever the bond becomes broken it must therefore be concluded that it was not sufficiently strong to withstand a shearing stress between the top and bottom layers of the amount thus computed. When it is recalled that such shearing stresses should be much more than equal to corresponding tensile ones, the inadequacy of much work is apparent, and the tests of the tensile strengths developed by the various methods of securing a proper bond tend to prove that a sufficient bond can be secured in floor and sidewalk work with proper workmanship, particularly if some special method like the acid treatment is used, in the very execution of which a sufficient number of special precautions are added to improve the result perceptibly.

The final experiment consisted in the fabrication of a reinforced concrete beam, doing the concreting in sections, each succeeding part being deposited after a lapse of about a week from the last one, while the beam was tested after a final lapse of only two weeks after the last section was concreted. The several parts and the order of deposit were as shown in Fig. 1. The mixture used was 1: 2: 4 (by weight), the broken stone being the run of the crusher, up to ¾-in. One of the bonding powders (Bonsit) was used to better the connection between successive parts. The beam was tested, on a 6-ft. span, with a center

load, the first application, up to 2 500 lb., being applied at the neutral axis on two projecting rods which were cast in the beam for the purpose. The concrete section was 8 by 4 in. over all, reinforced with two ½-in. smooth round rods placed ½ in. above the bottom of the beam, each one extending from one end to a point about 1 ft. from the other point of support. Each end was bent upward in a comparatively

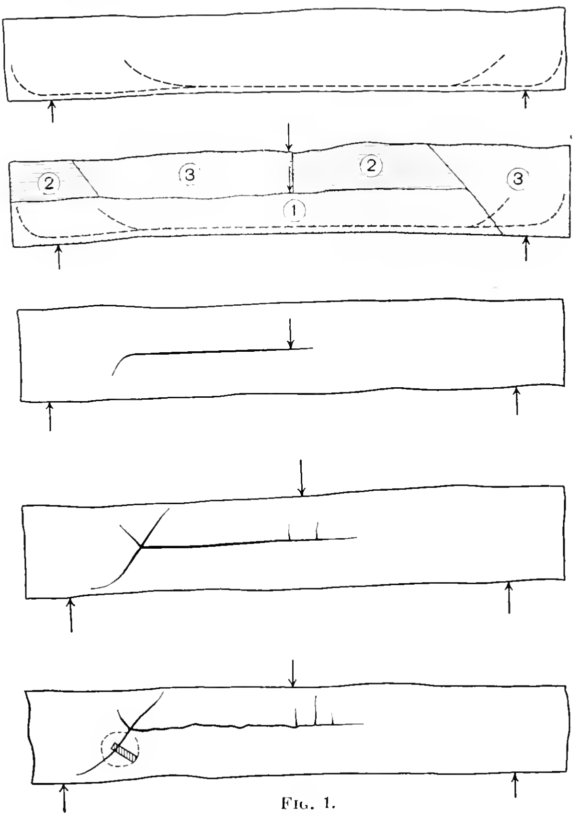

FIG. 1.

large-radius turn, the one at the end through an angle of 90° and the other one through about 45°, so as to serve to some extent in resisting diagonal tensile stresses. The reinforcement aggregated 1.34% of the effective beam area. The beam weighed about 180 lb. The load was applied in increments of 500 lb. each, the effect of each being carefully noted.

The shapes of the sections were determined with the following objects in view:

(*a*) To determine the effect of a horizontal joint in a beam, even when bonded as well as possible, where no steel is used for the purpose, when the load is applied at the level of the joint. This is the condition which exists in beams which are designed to be as deep as the distance between vertically adjacent window heads and sills in a concrete building, and where the concrete is almost invariably deposited in two separate operations, with the joint between them occurring at the level of the concrete floor.

(*b*) To ascertain whether it were possible to secure, along surfaces theoretically close to those on which maximum diagonal tensile stresses are developed, a bond of sufficient strength to make it unobjectionable to stop work along such a surface.

(*c*) To ascertain whether there is any virtue in leaving a joint only along a surface theoretically close to those on which maximum diagonal compressive stresses are developed.

The test results obtained were as follows:

(*a*) At 2 500 lb., applied at the center, a horizontal crack appeared, extending from the load toward one end and turning sharply downward into the mass concrete close to the turned-up end of one rod.

(*b*) The load was then shifted to the top of the beam, whereupon the horizontal crack disappeared entirely and the diagonal one was barely visible.

(*c*) At 1 500 lb. applied on the top, a condition almost identical with (*a*) had been reached.

(*d*) At 3 500 lb., the diagonal crack had extended downward and was following along the reinforcing rod toward the end of the beam. Very small tension cracks had appeared in the upper part of the beam under the load, and a wedge-shaped piece was commencing to crack out at a point where the diagonal and horizontal cracks intersected, in the vicinity of, but not along, an artificial joint parallel with diagonal compression planes.

(*e*) The load gradually fell to about 3 000 lb., as more strain was produced, and all these cracks became more prominent, until a sliver of concrete was forced off, revealing the end of one rod cutting directly across the main diagonal crack.

The rod had slipped in the concrete near the end of the beam, but adhered firmly to that on the other side of the crack.

It is to be noted that the artificial joint, where diagonal failure

was expected to occur, was closer to one point of support than was the crack which actually occurred at the other end. Thus the diagonal stresses were larger on the artificial joint than they were where the mass concrete failed.

The results obtained in this particular beam may be summarized as follows:

(*a*) In a beam, a horizontal joint not reinforced with steel, even if carefully bonded with acid and grout, is such a cause of weakness that the beam should be designed of a height only up to the joint, whenever the load is applied at the level of this joint.

(*b*) In a beam, a joint can be made sloping in the direction of the cracks caused by diagonal tensile stresses, which will be as strong as mass work, if the joint is bonded in accordance with the specifications given above.

(*c*) In a beam, the concrete in the vicinity of a diagonal joint parallel with the lines of maximum diagonal compressive stresses will fail before a joint made oppositely does, if incipient failure has already occurred in the vicinity of what may be called the compression joint and has not done so near the tension joint.

As a general conclusion, it may then be stated, that if proper bonding methods are adopted, with acid and grout, work may be stopped along 45° planes at any point in a beam, at pleasure, between the ends and the third points. In view of the one other test reported besides the writer's, such planes would better be diagonal compression planes. The writer prefers such joints to vertical ones at the centers of beams, where direct tensile and compressive stresses are larger. Further, it is to be remarked that nothing but ample reinforcement, consisting of stirrups or numerous diagonal rods, can compensate for the detrimental effect of a horizontal joint left along or near the central axis of a beam. The writer prefers numerous stirrups, well connected to the tension rods, and amply anchored in the top concrete. In proof of the efficacy of this design, reference may be made to his brick beam experiment, and the one in which no concrete at all was placed around the tension rods.*

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 343.

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PAPERS AND DISCUSSIONS

STRESSES IN MASONRY DAMS.*

By William Cain, M. Am. Soc. C. E.

The object of this investigation is to determine the amounts and distribution of the stresses in a masonry dam, at points not too near the foundations, having assumed the usual "law of the trapezoid," that vertical unit pressures on horizontal planes vary uniformly from face to face.

Experiment indicates that such vertical stresses increase pretty regularly in going from the inner to the outer face, for reservoir full, until we near the down-stream or outer face, where the stress gradually changes to a decreasing one, which decrease continues to the end of the horizontal section. The law of the trapezoid is thus only approximately true over part of the section, but, as it gives an excess pressure where it attains a maximum, it errs on the safe side.

The profile of the dam selected is of the triangular type, with some additions at the top, but the method used in determining the stresses is general, and will apply to any type of profile. The final equations will give at any (interior or exterior) point of the horizontal section considered the vertical unit stress on the horizontal

* This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

section, the normal stress on a vertical plane, and the unit shear on either horizontal or vertical planes. From these stresses, the maximum and minimum normal stresses, and the planes on which they act, can be determined, and ultimately, if desired, the stress on any assumed plane can be ascertained.

The solution presented is approximate, which is justifiable, in view of the approximation involved in "the law of the trapezoid" used. The results, however, are practically correct, as will be evident from the checks applied, resulting from the exact theory given in the Appendix. The theory used, being simple, should be easily followed.

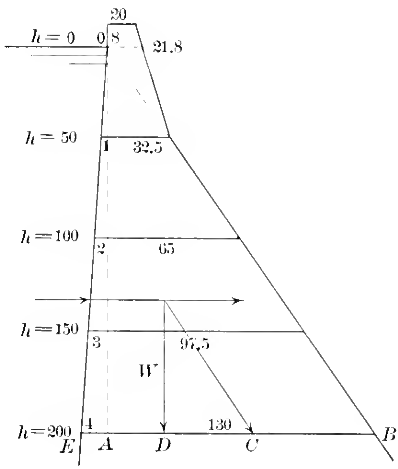

FIG. 1.

Let Fig. 1 represent a slice of the dam contained between two vertical parallel planes, 1 ft. apart and perpendicular to the faces. The batter of $O\,B$ is, $\frac{130}{200} = \frac{0.65}{1}$; that of $O\,E$ being $\frac{4}{200} = \frac{0.02}{1}$. The batter of the inner face was found by trial, so that the centers of pressure on horizontal sections, for reservoir empty, should nowhere pass more than a fraction of a foot outside the middle third of the section. The simple type of profile shown was adopted for ease of computation.

For convenience in subsequent computations, the breadth, $b = E\,B$, of horizontal sections, corresponding to various depths, h, below the

surface of the water in the reservoir, are given, all dimensions being in feet:

$$h = 199.0,\ b = 133.330$$
$$h = 199.5,\ b = 133.665$$
$$h = 200.0,\ b = 134.000$$
$$h = 200.5,\ b = 134.335$$
$$h = 201.0,\ b = 134.670$$

Take the weight of 1 cu. ft. of masonry equal to 1; then the weight of masonry above any section is equal to the corresponding area in Fig. 1 above that section. The area of the portion above $E\ O\ B$ is readily found to be 712, and its moment about the vertical, $A\ O$, is 11 603, the unit of length being the foot. In Fig. 1, D is where the vertical through the center of gravity of the dam above the joint, $E\ B$, cuts that joint, and C is the center of pressure on that joint when the water pressure on $E\ O$ is combined with the weight of masonry, W, above $E\ B$.

As h varies, suppose each horizontal joint, in turn, marked similarly to the joint at $h = 200$, with the letters E, A, D, C, B; then, for any joint, on taking moments of the triangles, $A\ O\ B$, $A\ O\ E$, and the area above $O\ B$, we find,

$$A\ D = \frac{\frac{A\ O}{6}\left(A\ B^2 - E\ A^2\right) + 11\ 603}{W}$$

Assuming that the masonry weighs $2\frac{1}{2}$ times the water per cubic unit, then the weight of a cubic foot of water is $\frac{2}{5}$. It would entail but little extra trouble here, where the inner face has a uniform batter throughout, to include the vertical component of the water pressure on the face, $E\ O$; but it will be neglected, as usual.

The horizontal water pressure for the height, h, is thus, $\frac{2}{5} \times \frac{h^2}{2} = \frac{1}{5}\ h^2$, and its moment about C is, $\frac{1}{5}\ h^2 \times \frac{1}{3}\ h = \frac{1}{15}\ h^3$.

Taking moments of W and water pressure about C, we have at once,

$$D\ C = \frac{1}{15} \times \frac{h^3}{W}.$$

From the last two formulas, we derive the following results:

h	W	$A\ D$	$D\ C$
199	13 978.335	40.49141	37.58483
200	14 112.000	40.70316	37.79289
201	14 246.335	40.91488	38.00089

A seven-place logarithmic table was used throughout, the aim in the computations being to get the seventh significant figure correct within one or two units. The necessity for this accuracy will be seen later.

The distances, $E\ C$ and $C\ B$, are now readily derived.

For, $h = 199$, $E\ C = 82.05624$, $C\ B = 51.27376$

$h = 200$, $E\ C = 82.49605$, $C\ B = 51.50395$

$h = 201$, $E\ C = 82.93577$, $C\ B = 51.73423$.

On any plane, $E\ B$, the vertical unit pressure,

$$\text{at } B = p_1 = \frac{4\ b - 6\ C\ B}{b^2}\ W,$$

$$\text{at } E = p_2 = \frac{4\ b - 6\ E\ C}{b^2}\ W;$$

where $b = E\ B$, and W is the weight of masonry above the plane. This follows from the assumed "law of the trapezoid."

From these formulas we derive,

At $h = 199$, $p_1 = 177.45483$, $p_2 = 32.22542$,

$h = 200$, $p_1 = 178.3855$, $p_2 = 32.24139$,

$h = 201$, $p_1 = 179.3160$, $p_2 = 32.25798$.

Call p the vertical unit stress at a distance, x', from E; then,

$$p = p_2 + \frac{p_1 - p_2}{b}\ x';$$

and the total stress on the base, x', is,

$$P = \frac{1}{2}\ [p_2 + p]\ x' = p_2\ x' + \frac{p_1 - p_2}{2\ b}\ x'^2 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

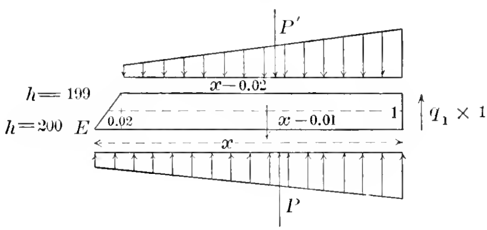

Fig. 2.

To find the unit shear on vertical or horizontal planes*, consider a slice of the dam, bounded by horizontal planes at $h = 199$ and

* The writer desires here to acknowledge his indebtedness to a recent paper on "Stresses in Masonry Dams" by Ernest Prescot Hill, M. Inst. C. E., published in *Minutes of Proceedings*, Inst. C. E., Vol. CLXXII, p. 134. Mr. Hill considers the case of a dam with a vertical inner face. By the aid of the calculus, he effects an exact solution, which leads to general formulas for shear and normal pressures on vertical planes.

Mr. Hill ascribes to Professor W. C. Unwin the suggestion, "that the shearing stress at any point may be found by considering the difference between the total net vertical reactions [between that point and either face] along two horizontal planes at unit distance apart," and states that Prof. Unwin "has applied the principle to a triangular dam by the use of algebraical methods."

$h = 200$, the water face and a vertical plane, at a distance, x, from the inner face (Fig. 2), in equilibrium under the water pressure acting horizontally on its left face and the forces exerted by the other parts of the dam on the slice. These forces consist of the uniformly increasing stress, P', on top, acting down; the uniformly increasing stress, P, on the bottom, acting up; a shear acting on the vertical plane at the right, of average intensity q_1 per square foot, the weight of the body $(x - 0.01)$, besides the horizontal forces to be given later. The vertical component of the water pressure is here neglected, as usual. The origin for x is taken, here and in all subsequent work, at the level, $h = 200$, at the inner face.

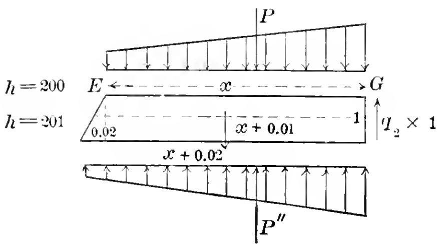

FIG. 3.

For equilibrium, the sum of the vertical components must be zero. Therefore, $q_1 = (x - 0.01) + P' - P$......................(2)

To find P', substitute in Equation 1, $x' = x - 0.02$, $p_2 = 32.22542$, $p_1 - p_2 = 145.22941$, $b = 133.330$, giving $P' = 32.20364\, x + 0.5446238\, x^2 - 0.6442906$. P, $x' = x$, $p_2 = 32.24139$, $p_1 - p_2 = 146.1441$, and $b = 134$; therefore

$$P = 32.24139\, x + 0.5453138\, x^2.$$

Substituting in Equation 2, we derive the average unit shear,

$$q_1 = -0.6542906 - 0.96225\, x - 0.0006900\, x^2 \quad ..(3)$$

This value of q_1 is strictly correct when $x \geqq 0.02$. It is slightly in error when $0 < x < 0.02$.

A similar investigation holds to obtain the average unit shear q_2 (Fig. 3) on a vertical plane, at a distance, x, from E, extending from the level, $h = 200$, to the level, $h = 201$.

We have, for equilibrium,

$$q_2 = (x + 0.01) + P - P'' \quad ....(4)$$

We find P'' by substituting in Equation 1, $x' = (x + 0.02)$, $p_2 = 32.25798$, $p_1 - p_2 = 147.05802$, and $b = 134.67$. $P'' = 32.27982\, x +$

$0.5459941\,x^2 + 0.6453780$. Substituting this, and the value previously found for P, in Equation 4, we derive,

$$q_2 = -\,0.6353780 - 0.96157\,x - 0.0006803\,x^2 \ldots\ldots (5)$$

This is strictly correct only when $x \geqq 0$.

The mean, $\frac{1}{2}(q_1 + q_2)$, of these average shears will be assumed as approximately equal to the intensity of shear at the point, G ($x = E\,G$), at the level, $h = 200$. Call q this intensity of shear on a vertical plane at G; therefore,

$$q = -\,0.6448343 + 0.96191\,x - 0.0006856\,x^2 \ldots\ldots\ldots (6)$$

Checks.—By Appendix (b) and (d), the exact value of q, at either face, $= p$ tan. ϕ, where $p =$ vertical unit normal stress at the face and ϕ is the angle the face makes with the vertical. Thus, at the inner face, $q = -\,32.24139 \times 0.02 = -\,0.6448278$, whereas Equation 6 gives, for $x = 0$, $q = -\,0.6448343$.

At the outer face, the exact value is, $178.3855 \times 0.65 = 115.9506$, whereas Equation 6 gives, for $x = 134$, $q = 115.9405$.

A still more searching test can be devised. It is a well-known principle that the intensity of shear at a point, on vertical or horizontal planes, is the same [Appendix (a)]. Therefore, regarding Equation 6 as giving the horizontal unit shear, at the level, $h = 200$, where $b = 134$ ft.; the total shear, from face to face, on this level, is,

$$\int_{x=o}^{x=134} q\,dx = 7999.75$$

This should equal the total water pressure down to the same level, $\frac{1}{5}(200)^2 = 8\,000$. Formula 6 thus gives practically exact results.

In order to find the normal unit stress on a vertical plane, we shall assume that q_1, given by Equation 3, equals the intensity of shear on a vertical or horizontal plane at the point x, at $h = 199.5$; and that q_2, given by Equation 5, gives the shear intensity at x at $h = 200.5$. This evidently supposes that the shear intensity increases uniformly, vertically, from $h = 199$ to $h = 201$.

Consider a portion of the dam, Fig. 4, bounded by the water face; the plane, $F\,M$, at the level, $h = 199.5$, on which the total shear is Q', the plane, $E\,N$, at the level 200.5, on which the total shear is Q, and the vertical plane, $M\,N$, 1 sq. ft. in area, on which the average normal stress is p'. The water pressure on $E\,F$ will be supposed to be exerted horizontally. It is equal to 80 units. Assuming, as stated, that $q_1 =$

intensity of horizontal shear at M, and q_2 = the corresponding intensity at N, we have, taking the origin as before O,

$$Q' = \int_{0.01}^{x} q_1 \, dx \, ; \qquad Q = \int_{-0.01}^{x} q_2 \, dx \, ;$$

or,

$$Q' = 0.006494794 - 0.6542906 \, x + 0.481125 \, x^2 - 0.00023 \, x^3$$
$$Q = -0.00640186 - 0.6353780 \, x + 0.480785 \, x^2 - 0.0006803 \, \frac{x^3}{3}$$

Checks.—The total water pressure for $h = 199.5$ is $\frac{1}{5}\,(199.5)^2 =$ 7960.05 and for $h = 200.5$, $\frac{1}{5}\,(200.5)^2 = 8\,040.05$. The first should equal Q', for $x = 133.665$, or 7 959.22; the second should equal Q, for $x = 134.335$, or 8 041.12. The slight differences tend to give confidence in the results.

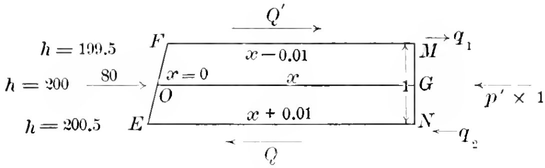

FIG. 4.

For equilibrium, the sum of the horizontal forces acting on $E\,F\,M\,N$, Fig. 4, must be zero;

therefore, $p' = 80 + Q - Q'$ .... (7)

$$p' = 80.01 - 0.0189 \, x + 0.00034 \, x^2 - 0.00000323 \, x^3.$$

This average stress will now be assumed to be the intensity of the horizontal unit stress on vertical planes at $h = 200$.

It will now be perceived why a seven-place table was necessary in the computations, the coefficients of x^2 and x^3 having only two or three significant figures in the final result. If the planes originally had been taken 0.1 ft. apart vertically, a ten-place table would have been required.

Checks.—The value of p', for $x = 0$, $p' = 80.012896$, is the same as that given by Appendix (a), $80 + 0.6448 \times 0.02$. When $x = 134$, the formula gives $p' = 75.81$, whereas the exact theory, Appendix (b), gives $p' = m^2 \, p = (0.65)^2 \times 178.39 = 75.37$. The difference is 0.44 at the outer face. For any other point, it might be assumed to vary with x, so that it could be corrected by subtracting $\frac{0.44}{134} x = 0.0033 \, x$

from the value of p' above. For ease of computation, the formula will be written,

$$p' = 80.01 - 0.02\,x + 0.00034\,x^2 - 0.00001\,\frac{x^3}{3} \quad \ldots\ldots (8)$$

The first coefficient of x^3 cannot be counted on to the last two figures, hence we are permitted to change 323 to 333 in that coefficient. When $x = 134$, Equation 8 gives $p' = 75.41$, nearly the exact value.

The three formulas for p, q and p', at the level $h = 200$, are thus as follows:

$$p = 32.24 + 1.09063\,x.$$

$$q = -0.64 + 0.962\,x - 0.000686\,x^2.$$

$$p' = 80.01 - 0.02\,x + 0.00034\,x^2 - 0.00001\,\frac{x^3}{3}.$$

Since the weight per cubic foot of masonry was assumed as two and one-half times that of water, we must multiply the stresses given in Table 1 by $\frac{5}{2}\,(62.5) = 156.25$, to reduce to pounds per square foot; or by 1.085, to reduce to pounds per square inch.

TABLE 1.

x	0	10	25	50	75	100	134
p	32.24	43.15	59.50	86.77	114.04	141.30	178.39
q	−0.64	8.91	22.98	45.75	67.65	88.70	115.95
p'	80.01	80.02	79.66	79 11	79.01	78.08	75.37
max. f	80.02	82.06	94.67	128.85	166.40	203.85	253.71
min. f	32.23	41.11	44.48	37.03	26.64	15.52	0
θ for max. f	90°46′	77°06′	56°50′	42°36′	37°44′	35°12′	33°01′

In Table 1 the stresses are those experienced at the level, $h = 200$.

p = vertical unit stress on a horizontal plane,

q = shearing unit stress on horizontal or vertical planes,

p' = horizontal unit stress on vertical planes,

Max. f = maximum normal stress acting on a plane inclined to the horizontal at the angle θ, given on the last line,

Min. f = minimum normal stress acting on a plane perpendicular to the last.

From max. f and min. f, with θ, the ellipse of stress can be drawn, and the stress in any direction, with the plane on which it acts, can be ascertained.

It will be observed that there is no tension exerted anywhere, and that the maximum compression is 253.71, or 275 lb. per sq. in., which

is exerted at the outer face, parallel to that face, upon a plane at right angles to the face.

In Appendix (*e*), the important formula, for the maximum normal intensity at the outer face, acting parallel to that face,

$$f = \frac{p}{\cos.^2 \phi}$$

is proved. In this instance, $p = 178.39$, tan. $\phi = 0.65$, therefore $\phi = 33°01'$, whence $f = 253.71$.

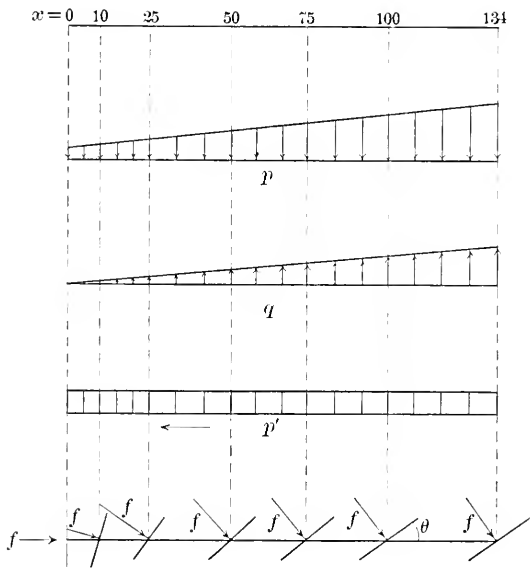

FIG. 5.

This stress is unaccompanied with any conjugate stress, perpendicular to the face. In the interior of the dam, where conjugate stresses prevail, the masonry is perhaps better able to withstand a certain compressive stress than at the face. The distribution of stresses, at the level $h = 200$, is shown in Fig. 5, on the supposition that the base of the dam is a little below that level. The connection with the foundation materially modifies this distribution; but Fig. 5 shows the distribution for sections, say, from 10 to 20 ft. above the base, up to the level $h = 50$, fairly well, on the basis of the trapezoid law. As has been mentioned before, this law gives a pressure greater than the actual at the outer face.

Since the batter of the inner face is very small, the results of Table 1 should agree approximately, except near the inner face, with those found by Mr. Hill in the paper referred to in the foot-note. Substituting numerical values, Mr. Hill's formulas, for $h = 200$, reduce to,

$$q = 0.9426\ x - 0.0005768\ x^2,$$
$$p' = 80 - 0.0001289\ x^2 - 0.0000009615\ x^3;$$

giving,

x	0	10	25	50	75	100	134
q	0	9.36	23.20	45.69	67.45	88.49	115.95
p'	80	79.99	79.90	79.56	78.87	77.75	75.38

On comparing these formulas with those of the writer, it will be observed that the absolute term in the value of q and a consequent term of the first degree in x, in the value of p', are lacking in Mr.

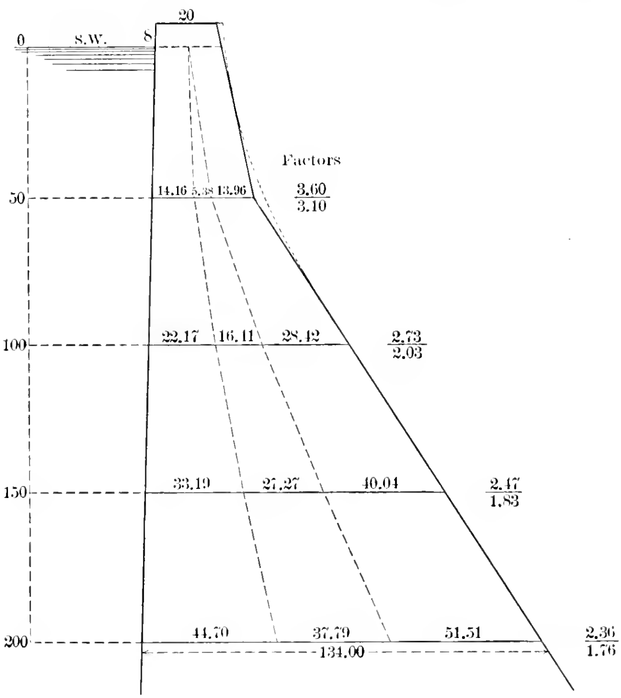

FIG. 6.

Hill's formulas. This results from taking the inner face as vertical. Although the coefficients also differ, it is seen that the numerical values are very nearly the same.

In Fig. 6 are shown, on a drawing of the dam, to scale, the lines of the centers of pressure for reservoir full and empty.

To the right, and under the word "factors," are certain numbers, written in the form of fractions. For any joint, the upper number gives the factor against overturning, or the number by which it is necessary to multiply the water pressure down to the joint, to cause the total resultant to pass through the outer edge of the joint considered. The lower numbers give the ratio of the weight of masonry above a joint to the water pressure corresponding.

It is believed that these "factors" should increase from the base upward, to allow somewhat for earthquakes, expansion of ice in freezing, etc., since the effect of such accidental forces is proportionately greater on the upper joints.

Stresses due to water infiltration are not included here; neither are stresses due to temperature changes.

The unit stresses, f, in pounds per square inch, acting parallel to the adjacent face, are as follows, and refer to the outer edges of the joints, for reservoir full, and to the inner edges, for reservoir empty:

h.	f at outer edge.	at inner edge.
50	85	58
100	136	133
150	204	180
200	275	228

The stresses, f, are normal pressures on planes perpendicular to the respective faces, and are the greatest stresses that can be experienced in the dam. In fact, they are greater than the true stresses, since the trapezoid law is not exact, particularly near the base, as before remarked. It would then seem that the dam, thus far, is safe, since the maximum unit stress is less than concrete, even, is subjected to daily, in good practice.

For an actual construction, the outer face should be curved, from near $h = 50$ to the top, as shown by the curved dotted line in Fig. 6.

The subject of the stresses in masonry dams has caused a great deal of discussion among British engineers in the last two or three years. The subject was reopened by Mr. L. W. Atcherly and Professor Karl

Pearson,* who gave the results of certain experiments which seemed to indicate considerable tension across vertical planes near the outer toe. The late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., also published† the results of experiments on a model dam of stiff jelly, and very recently, the "Experimental Investigations" of Sir J. W. Ottley and Mr. A. W. Brightmore‡ on elastic dams of "plasticine" (a kind of modeling clay) and the experiments of Messrs. J. S. Wilson and W. Gore§ on "India Rubber Models" have been presented.

It is not the object of this paper to discuss these later experiments; but it may be remarked that they show very plainly that no tension exists near the outer toe, but that tension does exist at the inner toe, where the dam is joined to the foundation, and it has become a serious matter how to deal with it. The influence of the foundation in modifying the distribution of the stresses at the base of the dam was found to be very great, causing the shear there to be more uniform than higher up, where the parabolic law, nearly as given by the formulas above, was found to hold. Also, above some undetermined plane, a small distance above the base, the usual "law of the trapezoid" was found to be approximately correct, leading to stresses on the safe side at the outer toe. This law leads to stresses at the outer toe of the base considerably in excess of the true ones.

It was found, from the rubber models particularly, as theory indicates, that the greatest normal pressures are exerted at the downstream face, for reservoir full, and they act in a direction parallel to that face.

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXII, p. 456.
† *Ibid*, Vol. CLXII, p. 123.
‡ *Ibid*, Vol. CLXII, p. 89.
§ *Ibid*, Vol. CLXII, p. 107.

APPENDIX.

Relations Between Stresses at any Point of a Dam.

(*a*).—Consider a cube of masonry, Fig. 7, the edge of which has the length a, bounded by vertical and horizontal planes and subjected to normal and shearing forces, caused by the action of the other parts of the dam. Since a will be supposed to diminish indefinitely, the weight of the cube, which is proportional to a^3, is an infinitesimal of the third order, and can be neglected in comparison with the normal forces, which vary as a^2 and are thus of the second order.

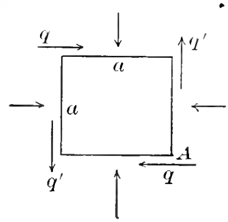

Fig. 7.

Similarly, the average unit stresses exerted on the faces can be treated from the first as the unit stresses at any point, A, of the cube. As a diminishes indefinitely, the oppositely directed normal forces approach equality and balance independently; hence the couples formed by the shears on opposite faces must likewise approach equality; the one being right-handed, the other left-handed; therefore $q\ a \times a = q'\ a \times a$, or $q = q'$; hence, the intensities of shear at a point on two planes at right angles are equal. The relative directions of the shears on two planes at right angles are determined, as above, from the consideration that one resulting couple must be right-handed and the other left-handed. This applies also to Figs. 8 to 11.

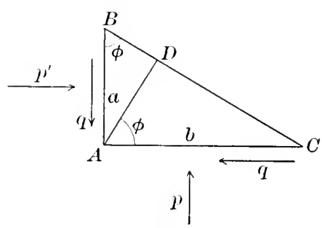

Fig. 8.

(*b*).—In Fig. 8, $A\ B\ C$ is the right section of a prism at the outer face, with lateral faces one unit in length, perpendicular to the plane of the paper.

Let $A\ B$ be vertical; tan. $\phi = m$, a constant;

p = normal intensity on a horizontal plane at C,

p' = normal intensity on a vertical plane at C,

q = shear intensity on horizontal or vertical planes at C.

The weight of the prism is, $\frac{1}{2}\ a\ b$.

Balancing vertical as well as horizontal components, we have, when $a = A\ B$ and $b = A\ C$ are very small,

$$p\ b = q\ a + \frac{1}{2}\, a\ b, \text{ nearly;}$$
$$p'\ a = q\ b.$$

Dividing the first equation by b, the second by a, the limit, as a and b approach zero, gives exactly,

$$p\ = q \text{ cot. } \phi, \text{ therefore } q = m\ p,$$
$$p' = q \text{ tan. } \phi, \text{ therefore } p' = m^2\ p,\ p\ p' = q^2.$$

These equations give the relations between p, q and p' at the outer face. The same relations hold at the inner face, for reservoir empty, on replacing ϕ by ϕ', the angle the inner face makes with the vertical.

For the remaining cases, the final limits will be written at once, since the complete process of deriving them is evident from the above. In fact, the weight of the prism, $\frac{1}{2}\ a\ b$, being of the second order, can be neglected in comparison, with $q\ a$, etc.

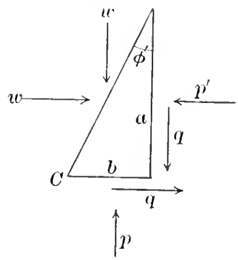

FIG. 9.

(c).—For reservoir full, calling $w = \frac{2}{5}\ h$, the intensity of water pressure, horizontally or vertically, at C, we have at the inner face, putting tan. $\phi' = n$, Fig. 9,

$$p\ b = q\ a + w\ b;\ p'\ a = q\ b + w\ a;$$

therefore $\quad p = \frac{1}{n}\ q + w;\ p' = q\ n + w.$

(d).—If the vertical component of the water pressure is neglected, these equations reduce to,

$$p = \frac{1}{n}\ q;\ p' = q\ n + w;$$

therefore $\quad q = p\ n,\ p' = n^2\ p + w.$

(e).—Since the shear on the outer face is zero, therefore, by (a), the shear on a plane, $A\ D$, Fig. 10, perpendicular to the outer face, is also zero, or the stress on $A\ D$ is normal.

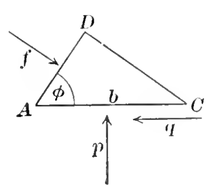

FIG. 10.

Call f the intensity of such a stress at C. The total pressure on $A\ D = f \times A\ D = f\ b$ cos. ϕ, and its vertical component is $f\ b$ cos.$^2\ \phi$, therefore balancing the vertical components,

$$p\ b = f\ b \text{ cos.}^2 \phi;$$

therefore $$f = \frac{p}{\cos.^2 \phi} = p \sec.^2 \phi$$

This is a most important formula for finding the maximum normal intensity at the outer face. It applies equally to the inner face for reservoir empty, on changing ϕ to ϕ', the angle the inner face makes with the vertical. For either face, p is the vertical normal unit stress at the face considered.

(*f*).—*Principal Normal Stresses at Any Point in the Dam and the Planes on Which They Act.*—In the prism, $A\ B\ C$, Fig. 11, let $A\ B$ be one of the planes on which the stress is normal. Let f be its intensity. The stress on the plane, $A\ B$, of unit length perpendicular to the plane of the paper, is thus $f\ c$; its vertical component is, $f\ c \cos. \theta = f\ b$, and its horizontal component is $f\ c \sin. \theta = f\ a$, θ being the angle that $A\ B$ makes with the horizontal.

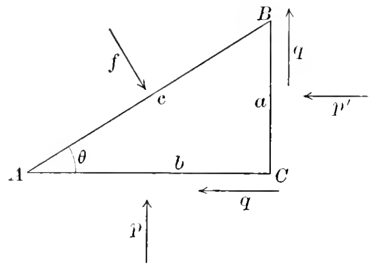

FIG. 11.

Place the sum of the vertical forces acting on $A\ B\ C$ equal to zero; also place the sum of horizontal forces equal to zero.

$$f\ b = p\ b + q\ a, \text{ therefore } f - p = q \tan. \theta,$$

$$f\ a = q\ b + p'\ a, \text{ therefore } f - p' = q \cot. \theta.$$

The difference of the last two equations gives,

$$p - p' = q\ (\cot. \theta - \tan. \theta) = q\ \frac{1 - \tan.^2 \theta}{\tan. \theta},$$

therefore $$\tan. 2\ \theta = \frac{2 \tan. \theta}{1 - \tan.^2 \theta} = \frac{2\ q}{p - p'}.$$

The angles, θ (differing by 90°), computed from this equation, give the directions of the planes, $A\ B$, on which the stress is entirely normal.

From an equation above, we likewise have,

$$\tan. \theta = \frac{f - p}{q}.$$

This gives directly the plane on which a given f acts.

To deduce a formula for f, take the product of two equations above.

$$(f - p)\ (f - p') = q^2.$$

therefore $$f = \frac{1}{2}\ [p + p' \pm \sqrt{(p + p')^2 - 4\ (p\ p' - q^2)}].$$

This equation gives the two values of f corresponding to the two planes mentioned; compressive when f is positive, tensile when negative. There can be no tension when $p\,p' \geqq q^2$.

A better form for computation is,

$$f = \frac{1}{2}\left[p + p' \pm \sqrt{(p - p')^2 + 4\,q^2}\right].$$

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE IMPROVEMENT OF THE OHIO RIVER.

Discussion.*

By Theron M. Ripley, Assoc. M. Am. Soc. C. E.

Mr. Ripley. Theron M. Ripley, Assoc. M. Am. Soc. C. E. (by letter).—A reading of Major Sibert's paper brings again the query which has come to mind many times in the past few years, viz., on what data and after how careful consideration of the questions has been based the assumption that a possible 11 ft. is the maximum which should be provided for on the Ohio improvement, and if 9 ft. is economically necessary at Pittsburg at present, would not 11 ft. or more be the economical development below Portsmouth or Cincinnati?

This query is not an insinuation as to the paucity of data or lack of study of the scheme for the Ohio River as a river, but in its relation to the possibilities and probabilities of contiguous improvements and their bearing on the Ohio River work.

The State of Ohio contains at least one, and maybe two, routes along which it is possible to construct a canal from Lake Erie to the Ohio River with a depth of water of not less than 12 ft.

For several years a determined effort has been made (and is now being made) by some of the State's best men to have such a canal constructed. There are those who believe that the State could expend no money which would be of greater economic benefit than in building such a canal.

Ohio is geographically situated directly between the immense ore deposits of Northern Michigan and the no less immense deposits of coal

* This discussion (of the paper by William L. Sibert, M. Am. Soc. C. E., printed in *Proceedings* for October, 1908) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

in West Virginia. Already traversed by the trunk lines of nearly all the railroads from the Atlantic to St. Louis and northern points, what more natural than that some of her citizens should believe in bringing this coal and iron together by the cheapest method, and shipping her manufactured product by her own waterway and the Barge Canal of New York State to New York City, or by her railroads to Atlantic and inland points farther south, or by her canal and river to New Orleans and intermediate points? Mr. Ripley.

Questions such as these should be taken into consideration in any development for the Ohio, as any structures in that river will determine the economic navigable depth of connecting waterways above, and may assist or destroy their usefulness. In fact, a less depth below Portsmouth or Cincinnati than that possible across the State of Ohio might prevent the building of an Ohio Canal, and in any event would be a serious handicap thereto.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

ELECTRIC RAILWAYS IN THE OHIO VALLEY BETWEEN STEUBENVILLE, OHIO, AND VANPORT, PENNSYLVANIA.

Discussion.*

By Messrs. F. Lavis, George B. Preston, J. Martin Schreiber, and William J. Boucher.

Mr. Lavis. F. Lavis, M. Am. Soc. C. E.—The valley of the Ohio River and the territory tributary to it have witnessed a greater development of long-distance interurban electric railroads than any other part of the United States. Many of these lines run through sleeping, dining and drawing-room cars, built by the Pullman Company, of only slightly lighter construction than the standard equipment of the largest trunk lines of steam railroads. A description, therefore, of one of these lines, completed only about six months ago by one of the most prominent engineering firms identified with work of this kind, is presumably a description of the very latest and best practice in the construction of railroads of this type at the present time.

The general characteristics of these interurban lines are, that they pass through the various cities and towns along the route on the surface of the streets, and as near the business centers as possible, while in the open country between, they are located, for the greater part of the distance, on private right of way, in many instances having numerous highway crossings at grade. This latter feature is not mentioned by the author, and may have been avoided on this road, but the

* This discussion (of the paper by George B. Francis, M. Am. Soc. C. E., printed in *Proceedings* for November, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

number of these crossings, which are only slightly, if at all, less Mr. Lavis. dangerous than those of steam railroads, is becoming increasingly large, even in States where a great deal of public money has been, and is being, expended in eliminating such features on steam railroad lines.

This road, between Rochester and Steubenville, passes through a territory described by the author as a very "hive of industry," and the population tributary to it, amounting to more than a million people, or practically 100 000 per mile on the whole length between Pittsburg and Wheeling, is composed largely of the class of prosperous mechanics and laborers which furnishes a larger patronage, in proportion to the total population, to a road of this kind than does any other. The most substantial construction, therefore, is apparently warranted.

In order to get some idea of the relative importance of the construction on such a line, it seems desirable to have some basis of comparison with other standard railroad practice. The bridges on this line are designed for a concentrated load of 24 tons on two axles at 10-ft. centers, or 1 800 lb. per lin. ft. of single track. The heaviest equipment of the New York Subways and Elevated Lines has a load of 30 tons on the two axles with the motors, and 11 tons on the other two, or an average of 2 080 lb. per lin. ft. of track, which is only slightly greater than that used on this line. The largest Pullman cars in use on steam railroads are about 80 ft. long, and weigh about 60 tons, this load being carried on two six-wheeled trucks, giving about 10 tons per axle and about 1 500 lb. per lin. ft. of track. A passenger locomotive of the Atlantic type, weighing about 75 tons, which is a fairly large engine for express passenger service on anything but mountain grades, has a load of about 20 tons per axle on the driving wheels, and the load, for engine and tender loaded, will be about 4 000 lb. per lin. ft. of track. A consolidation freight engine, weighing about 100 tons, has a load of about 22 tons on the driving axles, and will average 5 000 lb. per lin. ft. of track. The bridges on this road, therefore, will carry almost any standard railroad equipment except locomotives, the weights of the heavier types of which in ordinary use would be about double that for which these bridges were designed.

Having in view, therefore, the fact that the requirements of roads of this class are not, in many respects, very far removed from standard steam railroad practice on first-class lines, and are in some respects greater than the requirements on many inferior lines, it has seemed to the speaker that it would greatly enhance the value of the paper if the author would give some of the reasons for the very radical departures from steam railroad practice as regards the location of this line.

The author states that "It required considerable engineering study and skill * * * to locate a line * * * which would give easy curves and grades," considering the natural difficulties of topography,

Mr. Lavis. and the possibility of securing private right of way. Nothing is stated in the paper in regard to what the author considers to be easy curves for a road of this type, although, as a matter of fact, outside of the cities, curves of from 200 to 300 ft. radius, provided there were not too many of them, and the amount of central angle covered was necessary, would probably have little effect either on cost of operation or speed. In passing through cities, curves of as small a radius as 50 ft. are not infrequent. This difference between roads of this type and trunk-line railroads where a curve of 1 000 ft. radius is considered quite sharp and necessitates slowing down to from 20 to 25 miles an hour, is quite marked, and is due of course to the absence on the electric road of the long rigid wheel base of the steam locomotive.

The question of grades on these electric roads, however, is one about which far less is known, at any rate publicly. Railroads building new lines to be operated by steam, even in a great deal of the, as yet, undeveloped parts of the country, are very reluctant to adopt higher rates than 0.6% unless the topography of the country absolutely demands it, whereas on the profile of the road under consideration, as shown by Fig. 2, one grade of 9.9% is shown for a fairly long stretch, and there are several stretches of 6% to 7%, although most of them seem to be fairly short.

The reason, of course, for the badly broken grade line is the fact that the river bank is occupied by the steam railroad, thus forcing the electric road in many cases high up on the side-hills where these come close to the river and necessitating its descent nearly to the river level where the hills lie farther back, in order to avoid a long detour to keep up. This characteristic of the country is shown very clearly in the view of the Yellow Creek Bridge.

The ability to ignore, to a very large extent, considerations which affect the determination of the rates of grade, which govern steam railroad practice, is the chief characteristic which differentiates the location of interurban electric railroads, or any electrically operated railroad where single cars or multiple-unit trains are to be used exclusively, from that of those operated by steam or electric locomotives at the head of the train, although few data have as yet been made public as to the actual conditions which govern the determination of the most economical grades to be used on these electric roads.

On a steam railroad, in conjunction with the necessity of low grades, it is also almost as important to arrange the grade line so that the demand on the locomotive will be as nearly even as possible throughout comparatively long stretches. No considerations of this kind apparently influenced the fixing of the grades on this railroad, and here again it would undoubtedly be of considerable interest to know just what considerations governed the judgment of the engineers in adopting the grades they did on this particular road.

As the steam locomotive is most economically operated when the demand on the machine is fairly consistent, so also is this the case with almost any machine or power plant, including, and perhaps especially so, electrical power plants. Mr. Lavis.

In the case of the latter, when power is being manufactured for the operation of a railroad, it may be considered that, where a fairly large number of cars are being operated, the demand will be equalized by the fact that some cars will be going down grade while others are going up, some stopped while others are going at full speed, etc.; but, on a road such as the author describes, where the service is infrequent and the number of cars in use at a time small, the very broken grade line and the high rates of grade would apparently tend to create a somewhat irregular demand, as there would be nothing to prevent a combination frequently occurring where all the cars, or at least a majority of them, would be going up hill at once, and then within a few minutes be all going down hill. In the case of power-houses supplying power only for the operation of the cars on a railroad of this kind, this might be a serious consideration. The power-houses on this line, however, supply power for both manufacturing purposes and electric lighting, for which purposes the load is probably fairly even, and is a large proportion of the total output. The variation in the load from the operation of the cars, therefore, probably does not affect them greatly. The great economy of a uniform load at a power-station is strikingly exemplified by the contract recently made by the Commonwealth Edison Company, of Chicago, for supplying power to the street railway companies of that city.* The very low rate of 0.4 cent per kw-hr. was made, provided the demand was uniform, any variation from this uniform demand being penalized according to a certain schedule mutually agreed upon. Of course, a street railway operating in any large city will have high peak loads during the rush hours, and therefore the demand cannot be regular; this instance, however, being so recent, and the contract so important, serves to emphasize the desirability of uniform loads at the power-houses.

It is well to bear this in mind and not lose sight of the fact that the electrical operation of railways has its limitations, although many liberties may be taken with the grades and alignment that would not be permissible on steam railroads.

There is another question which it has seemed to the speaker might naturally occur to some of the readers of the paper, and that is: Before deciding on the construction of these lines at all, was any consideration given to the possible effect on the new line, should the steam railroad, with its truly easy grades and curves, decide to electrify its line and run short trains at frequent intervals, or even equip its line with one of the many types of motor cars now apparently in successful operation on many steam railroad lines, where frequent

* *Engineering News*, December 10th, 1908.

Mr. Lavis. service for a comparatively small number of passengers at a time is demanded? Probably the steam railroad would be at a disadvantage by its inability to reach the business centers of the various towns, and also to leave every passenger nearly at his own front door. It is quite probable, also, that such a service as that necessary to compete successfully with the electric lines might interfere too much with the through business of the road, both passenger and freight.

In constructing an interurban railroad across country, some years ago, where 60-ft. rails were used, the speaker had some trouble from the excessive expansion and contraction in such comparatively long lengths, until the ballast was filled in as close as possible to the top of the rails and on both sides of them. If the temperature changes are at all large, and the joints are left open sufficiently to prevent the track from kinking in hot weather, there is likely to be excessive pounding of the ends of the rails at the joints, due to the wide spacing. It would be interesting to know whether any such trouble has been found on this road; and, if so, what means were taken to overcome it. The standard cross-section of the track, shown in Fig. 3, seems to show that no ballast was placed above the tops of the ties.

It is quite common practice now, in laying street railway tracks through paved streets, to lay a fairly substantial concrete foundation both under and around the ties, and to such a height over them as will just allow for the type of paving to be used, with the necessary sand cushion under it. The speaker had occasion to note only a short time ago the installation of this type of construction for a street railway track operated by a large public service corporation, on a double-track line, where the cars were operated on 20 min. headway, with little if any special excursion business, so that the traffic was probably not greater than on the line described by Mr. Francis. No mention of such a type of track construction is made in the paper. If it was not used through the paved streets in the cities, was it on account of the necessity of curtailing expenses, or because it was not considered economical, in view of the expected traffic on the road? The excessive wear and tear on the motors of electric cars due to poor track has led to the very general adoption of the very best type of track construction possible for street railroads almost everywhere.

The width between centers of tracks, adopted on this road, namely, 9 ft. 8½ in., is the general practice on street railway work in cities, though many interurban roads, where high speed is used, have adopted a wider spacing through the open country. On the road recently completed between Baltimore and Washington the tracks were laid 11 ft. on centers, and 80-lb. rails in 33-ft. lengths were used. The fact that the bridges on the Ohio road were designed to allow the tracks to be spaced 12 ft. on centers allows the inference that the engineers had in view the possible necessity of wider spacing of the tracks in the future.

There is another point which the speaker believes is not made quite clear in the paper, and that is the reason for building a double-track road. The only information as to the amount of traffic is the statement that cars are operated on 30 min. headway for ordinary traffic, and that there is a considerable excursion business. On roads with cars operating normally on 30 min. headway, it is generally quite satisfactory to build a single-track road with sufficient sidings to provide for operation on half that headway, the speaker having in mind a road in New England which handles quite successfully a very large summer excursion business on this basis. Mr. Lavis.

The construction of steam railroads is now carried out along lines which have become fairly well standardized, and the type of construction necessary to meet certain conditions is in a general way well known to engineers at all familiar with this branch of the profession. The construction of electric railroads, however, owing to the much greater flexibility of the application of power, is subject to much greater variations, and a most intimate co-relation of the work of the engineers responsible for the location of the line and the electrical engineers is necessary in order to produce the most economical results. The practice in the construction of this road, as described by Mr. Francis, is so radically different from steam railroad practice that the speaker believes that a fuller statement of the reasons which led to the final design, so to speak, of this road, would be most interesting, not only to the members of this Society, but to all engineers interested in work of this class.

George B. Preston, Esq.—As having some bearing on the relation between the average and maximum loads on the power stations, it may be of interest to note that with the heavy holiday load of July 4th, 1908, the 1-hour maximum at the East Liverpool house was 2 000 kw., while the average load during the heaviest part of the day, from 7 to 11 p. m., was 1 800 kw. On an ordinary day the 24-hour average for this station is 850 kw. Mr. Preston.

When considering the question of peak loads, in connection with this system, it should be borne in mind that these two power-houses are furnishing all the commercial and city lighting, as well as carrying a considerable commercial motor load, for four cities having an aggregate population of 73 000.

Mr. Boucher has asked whether or not the possibility of using the third-rail was considered in connection with this railway system. It was not considered feasible to use the third-rail system because of the necessity of utilizing the local electric railways at Steubenville and East Liverpool which were already installed and equipped with the overhead trolley, and also from the fact that a considerable portion of the roadbed lies in public streets and highways.

Mr. Schreiber. J. MARTIN SCHREIBER, ASSOC. M. AM. SOC. C. E. (by letter).—Although no new engineering features, other than represented in modern practice, are brought out in the paper, it is an intelligent and comprehensive description of a very interesting electric railway.

It is gratifying, at least to those who have had experience in the maintenance of some of the old-time electric railways, to know that the capitalist is rapidly realizing that it even pays to spend some money on the permanent improvement of street railways. Indeed, the very near future will see almost all electric roads constructed with as much skill and forethought as is represented on steam roads. This will be necessary for economical, satisfactory, and profitable operation, and in order to compete with other carrying companies.

The physical difficulties encountered in building the track and roadway of the Ohio Valley lines were out of the ordinary, as the location was not favorable for the work, and the author seems to be justified in the assertion that the configuration of the country through which the road was built is such as practically to preclude the location and construction of a future competing line. No doubt this condition accounts for the variable and at some places severe grades of the track. However, since the road is double tracked, successful and satisfactory operation is facilitated.

The author states that 85-lb. rails, in 60-ft. lengths, and of the Am. Soc. C. E. section, were used throughout the construction and reconstruction. The writer is of the opinion that rails in 60-ft. lengths are not altogether desirable on open railway work. It is true that there are fewer joints and more of a continuous track with a longer rail, but there is a disadvantage in handling and difficulty in maintaining proper alignment. Unless special expansion joints are used, 60-ft. rails will kink in summer and have open joints in winter. The 33-ft. rail seems to work to the best advantage for open track, and the 60 and 62-ft. lengths for city or paved track.

From one of the photographs it appears that the rail in paved city streets is also of the T-section. The advisability of using the T-rail in paved streets and adopting it as a standard in all street railway work has been strongly advocated by many railroad men. The Way Committee of the American Street and Interurban Engineering Association will make this subject its principal work during 1909, and it is hoped that a definite recommendation will be offered at its next annual convention.

A number of authorities, including the engineers of cities, are co-operating with the street railways in permitting the installation of T-rails in the streets. Several of the up-State New York cities, including Utica and Syracuse, laid T-rails during 1908, and they are reported as very satisfactory; and such cities as Minneapolis, with T-rails and brick paving, claim to have some of the finest street-railway track and roadway.

The great advantage of the T-rail, over any other section, from an operating standpoint, and especially with the increasing wheel loads, is too well known to explain at length, and the question is often only in regard to permission from the authorities for its installation, as they generally object on the ground that the rail does not allow proper paving facilities, and that the paving wears too rapidly along the gauge line of the rail. Mr. Schreiber.

It is indeed unfortunate that in the early installation of T-rails in city streets, the paving, especially adjoining the rail, was not properly executed. This placed the T-rail in disrepute for municipal work, so that it is not uncommon for the engineer of a small borough to demand the installation of a Trilby rail, similar to that required in Philadelphia or New York. The writer has often found this to be the case where vehicular traffic was practically nil, and where the paving was macadam. Here the T-rail would not only be cheaper and a great deal more desirable by the railroad company than any other of the common girder sections, but would give better service and more satisfaction to the public at large. This paving about such a rail is as easily maintained as that adjoining the Trilby rail, and with the Trilby section the broken stone is continually getting into the groove and breaking wheel flanges, causing uncomfortable riding, disagreeable noises, and frequent derailments, not to mention the trouble of maintaining the line and gauge.

On account of the great improvement made in T-rail track construction, it is the consensus of opinion of railway engineers that it is only a question of time when municipal authorities will co-operate with railway companies in the adoption of the T-rail, except in the very largest cities, where the streets are narrow, and traffic is very heavy. Just how far the T-rail may be used in densely populated cities, and with heavy wagon traffic and narrow streets, where trucks are compelled to follow the tracks, is difficult to anticipate; but, for ordinary city and interurban railways the T-rail should certainly displace the girder section.

The great difficulty of maintaining track and keeping it in line, and with proper joints, with the Trilby and tram sections, are objections which must be overcome; and there are railway managements which, in all probability, will direct their efforts in the future to satisfying the municipal authorities, by building better track, with substantial foundation, proper drainage, and more improved pavements to suit the T-rail, rather than attempt to operate with an unsatisfactory rail that will meet the present paving conditions.

WILLIAM J. BOUCHER, ASSOC. M. AM. SOC. C. E.—This paper is interesting in several particulars, and draws one's attention to the very extended development of cross-country electric roads competing with steam roads in Ohio and Indiana, many of which publish time-tables and schedules, run "limited" trains, and traverse distances from 50 to Mr. Boucher.

Mr. Boucher. 100 miles on one division. As these roads are primarily to connect centers of population, they necessarily run through sparsely settled districts, and frequently on private right-of-way; the query then arises, why are so few roads operated by the "third-rail"?

In any consideration of the cost of installation, it is necessary to specify the kind of work wanted. Single-track, span-wire trolley construction may be had at a variety of costs, but, for similar operating conditions and equivalent current capacities, there is a marked difference in the cost of overhead and third-rail construction. Assuming, then, that first-class construction in both cases is called for; also similar operating and current conditions; a double-pole, single overhead trolley with feed wire; other material and labor necessary for erection may be had at a cost of from \$4 600 to \$5 000 per mile, depending considerably on the nature of the ground in which the poles are placed. An equivalent third-rail system, using an 80-lb. rail on extended ties and insulators, with splice-plates, bonds, and underground cable for road crossings, will cost, erected, about \$3 800 per mile of single track. The third-rail is unprotected. Board protection, similar to that installed in the New York Subway, will cost about \$2 000 per mile. The maintenance charges should be considerably less for the third-rail system over a period of years.

Noticing, then, this difference of \$800 to \$1 200 per mile in favor of the third-rail system, there must be some compelling reasons why the overhead trolley is used so much more frequently. Snow should not cause trouble to a third-rail if the interval between cars is no greater than 30 min., and during a storm any road must be prepared to run all night in order to keep the road open. Sleet, however, may cause trouble, but a scraper has been devised and is in use in New York which seems to overcome the difficulty. Of course, in the case of roads operating through country and towns, the frequent shifting of shoe and trolley would prove a nuisance, especially if a double-throw switch must also be operated, and without the latter the shoe is dangerous to passengers who congregate around the steps and near the trucks while waiting to board the cars. In the case of long-distance roads, however, with expanses of country and few towns, and running alongside a platform at stations, the third-rail would seem to be superior. If the road on its private right-of-way is properly fenced, and cattle guards are used at crossings, but little trouble will be caused by cattle and horses.

The following roads are now operating with the third-rail: Albany and Hudson; Wilkes-Barre and Hazleton; Lackawanna and Wyoming Valley; Aurora, Elgin and Chicago; Scioto Valley, Ohio.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

MARTIN WILLIAM MANSFIELD, M. Am. Soc. C. E.*

DIED SEPTEMBER 25TH, 1908.

Martin William Mansfield was born at Ashland, Ohio, on November 19th, 1850. His father, Martin H. Mansfield, was of English birth; his mother was Anna Saiger, of Mifflin, Pennsylvania.

Mr. Mansfield was graduated from Rensselaer Polytechnic Institute with the Class of 1871. In September of the same year, he entered the service of the Pennsylvania Lines West of Pittsburgh as Assistant Engineer in the Maintenance-of-Way Department on the Cumberland and Muskingum Valley Railroad (a subsidiary line), at Zanesville Ohio. He was promoted successively to Engineer of Maintenance of Way, Superintendent, and Assistant Chief Engineer, which position he held at the time of his death.

On June 24th, 1878, Mr. Mansfield married Miss Carrie Sampsell at Ashland, Ohio, who survives him, with their son, Sampsell W. Mansfield, and daughter, Miss Corinne S. Mansfield.

As a student at Troy Mr. Mansfield was diligent, earnest, and successful. One of his classmates writes of him, "he gave evidence at that time of the unusual talent, that crowned his later years, for working out difficult and abstruse mathematical problems." This talent was indeed characteristic of the man, and was frequently called into play by special lines of investigation assigned to him by his superior officers, who recognized his ability to analyze a mass of apparently heterogeneous facts, reduce them to order, and find the underlying fundamental principle.

Kindly, affable, and accessible, but strict in dicipline, he commanded the esteem and good-will of his subordinates. Earnest, conscientious, and upright, in all things, he had the confidence of his superiors. Quiet and unassuming in manner, cheerful in disposition, and equable in temper, he won the respect of all who came in contact with him.

Mr. Mansfield was elected a Member of the American Society of Civil Engineers on July 5th, 1882, and by his death the Society loses one whose professional abilities and private character were an honor to it.

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* Memoir prepared by Thomas H. Johnson, M. Am. Soc. C. E.

MARK WILLIAM SCHOFIELD, M. Am. Soc. C. E.*

Died November 27th, 1908.

Mark William Schofield was born in Smithfield, Rhode Island, on November 10th, 1846. His parents were of English stock, and came to the United States about the year 1844. His father died while he was very young, and his mother some years later, in 1864. During his early life he attended the village school in Georgiaville, and later, the Lapham Institute, in Scituate, an advanced academy where many men prominent in later life received a large part of their education.

While attending school his energetic nature led him to spend a portion of his time in the mill in his native town, where he acquired much practical information regarding the details of cotton machinery. His tastes, however, led him toward the profession which he afterward pursued, and early in 1867, and previous to July, 1868, he was for a time with Mr. William S. Haines, and with Cushing and DeWitt, two of the older surveying and engineering firms of Providence. In July, 1868, Mr. Schofield went West, and was engaged on the surveys of the Cairo and Vincennes road, with which General Ambrose E. Burnside was at that time closely identified. Desmond FitzGerald, Past-President, Am. Soc. C. E., was then in charge of the party of which Mr. Schofield was a member.

Late in the fall of 1869 he returned to Rhode Island and re-entered the office of the late Samuel B. Cushing, Sr., M. Am. Soc. C. E., in Providence, where, with the exception of about two years spent on the Northern Pacific Railroad, he remained until the death of Samuel B. Cushing, Jr., M. Am. Soc. C. E., which occurred in 1888. He then carried on the business as the successor of the Cushings, until his death in November, 1908.

In 1869 and 1870 he was leveler on the preliminary survey for the Milford and Lowell Railroad, of which the elder Mr. Cushing was Chief Engineer. In 1873-1874 he was Engineer in charge of the construction of the East Providence branch of the Providence and Worcester Railroad. In the spring of 1881 Mr. Schofield again went West, and, until September, 1882, was Assistant Engineer on the Yellowstone Division of the Northern Pacific Railroad, his section lying between Billings and Miles City, Montana. Here he served with great credit, and, by his faithful and painstaking work, won the confidence and respect of those in the Chief's office. After the completion of the Northern Pacific Railroad, Mr. Schofield resided in Providence and conducted a conservative engineering business of a general nature, doing active work up to within three weeks of the time of his death.

*Memoir prepared by W. H. G. Temple, M. Am. Soc. C. E.

His whole life was marked by that sterling character, unswerving honesty, and strict loyalty to the interests of his clients, which won the respect of all with whom he had either business or social relations, and he unquestionably filled the essential requirements of an honorable man.

Mr. Schofield was married on December 18th, 1873, to Annie S. Brown, a descendant of Chad Brown, one of the early landowners of Providence. His widow, together with four children, survives him.

Mr. Schofield was elected a Member of the American Society of Civil Engineeers on May 1st, 1907.

William V. Morse

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ON RAIL SECTIONS: Joseph T. Richards, C. W. Buchholz, E. C. Carter, S. M. Felton, Robert W. Hunt, John D. Isaacs, Richard Montfort, H. G. Prout, Percival Roberts, Jr., George E. Thackray, Edmund K. Turner, William R. Webster.

ON CONCRETE AND REINFORCED CONCRETE: C. C. Schneider, J. E. Greiner, W. K. Hatt, Olaf Hoff, Richard L. Humphrey, Robert W. Lesley, J. W. Schaub, Emil Swensson, A. N. Talbot, J. R. Worcester.

ON STATUS OF METRIC SYSTEM: Stacy B. Opdyke, Jr., D. A. Molitor.

ON ENGINEERING EDUCATION: Desmond FitzGerald, Benjamin M. Harrod, Onward Bates, D. W. Mead, Charles Hansel.

ON STEEL COLUMNS AND STRUTS: Austin L. Bowman, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, Lewis D. Rights, George F. Swain, Emil Swensson, Joseph R. Worcester.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER......................5913 Columbus.
CABLE ADDRESS........................"Ceas, New York."

Vol. XXXV. FEBRUARY, 1909. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

FIFTY-SIXTH ANNUAL MEETING.*

January 20th, 1909.—The meeting was called to order at 10 A. M.; President Charles Macdonald in the chair; Chas. Warren Hunt, Secretary; and present, also, about 250 members.

The reading of the minutes of January 6th, 1909, was dispensed with.

Messrs. George W. Parsons, Shirley C. Hulse, and Herbert E. Cantwell were appointed tellers to canvass the Ballot for Officers for the ensuing year.

The Annual Report of the Board of Direction, and the Annual Reports of the Secretary and of the Treasurer,† for the year ending December 31st, 1908, were presented and accepted.

* A full report of the Fifty-sixth Annual Meeting is printed on pages 54 to 70 of this number of *Proceedings*.

† For these reports, see pages 9 to 17 of *Proceedings* for January, 1909 (Vol. XXXV).

The Secretary reported that, in accordance with the unanimous report of the Committee to Recommend the Award of Prizes, the Board of Direction had awarded the prizes for the year ending with the month of July, 1908, as follows:

The Norman Medal to Charles C. Schneider, Past-President, Am. Soc. C. E., for his paper entitled "Movable Bridges."

The Thomas Fitch Rowland Prize to Edward E. Wall, M. Am. Soc. C. E., for his paper entitled "Water Purification at St. Louis, Mo."

The Collingwood Prize for Juniors to D. W. Krellwitz, Jun. Am. Soc. C. E., for his paper entitled "Reinforced Concrete Towers."

The Committee also reported in favor of taking some official notice of the paper entitled "Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions," prepared by sub-committees of the San Francisco Association of Members of the American Society of Civil Engineers, and the Secretary was directed to express to the members of that Association the appreciation of the Board and of the Society of the interesting and valuable papers presented.

The Secretary announced the decision of the Board of Direction to hold the next Annual Convention at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to 9th, 1909, inclusive.

The Secretary announced the appointment of the following members on the Special Committee to consider and report upon "The Design, Ultimate Strength, and Safe Working Values of Steel Columns and Struts": A. L. Bowman, Lewis D. Rights, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C. Osborn, George H. Pegram, George F. Swain, Emil Swensson, and Joseph R. Worcester.

A. L. Bowman, Chairman of the Special Committee on Columns and Struts, submitted a preliminary report,* containing a resolution to the effect that the United States Government be requested to make a sufficient appropriation for and proceed with the construction of a testing machine for full-sized bridge members, and that the Secretary be directed to forward copies of the resolution to the President of the United States, the Vice-President, and the Speaker of the House of Representatives.

The report was accepted and the resolution adopted.

The following were appointed members of the Nominating Committee to serve two years:

Frederick W. Gardiner..	*Representing District No. 1.*
Richard A. Hale.......	" " " *2.*
Edward B. Codwise.....	" " " *3.*
Frank Sutton..........	" " " *4.*
H. E. Riggs............	" " " *5.*
Charles C. Wentworth.	" " " *6.*
C. E. Fowler..........	" " " *7.*

* See page 56.

The Secretary presented a report from the Special Committee on the Metric System.*

The report was accepted and ordered printed, and the Committee was discharged with the thanks of the Society.

The Secretary presented Majority and Minority reports from the Special Committee on Concrete and Reinforced Concrete.†

It was ordered that the Majority and Minority Reports be printed for distribution to the membership, that they be made an order of business for discussion at the next Annual Convention, and that the Committee be continued.

The Secretary presented a progress report‡ from the Special Committee on Uniform Tests of Cement.

The report was accepted and the Committee continued.

The Secretary presented a progress report§ from the Special Committee on Rail Sections.

The report was accepted and the Committee continued.

The Secretary presented a letter|| from the Chairman of the Special Committee on Engineering Education.

The Secretary presented a report¶ from the Committee appointed at the Denver Convention to consider and report upon certain amendments to the Constitution, proposed by Mr. Clemens Herschel and others, which recommended that the amendment be "re-worded by the Secretary to apply to the Constitution as it now exists, and sent out to letter-ballot with an accompanying statement that it must be voted upon by letter-ballot to meet the requirements of the Constitution; but that in view of the adoption of an amendment since its presentation placing the election of members in the hands of the Board of Direction, it is the opinion of this Committee that it should be defeated."

The report was accepted and its recommendations adopted.

The Secretary made some announcements in reference to the programme of the Annual Meeting.

Samuel Whinery, M. Am. Soc. C. E., presented the following preamble and resolution:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

* This report was printed in *Proceedings*, Vol. XXXIV, page 415 (October, 1908).

† See page 85.

‡ See page 71.

§ See page 61.

|| See page 63.

¶ See page 64.

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

The resolution was adopted.

The report of the tellers* appointed to canvass the Ballots for Officers for the ensuing year was presented.

The President announced the election of the following officers:

President, to serve one year:

ONWARD BATES, Chicago, Ill.

Vice-Presidents, to serve two years:

GEORGE HERNDON PEGRAM, New York City.
EMIL SWENSSON, Pittsburg, Pa.

Treasurer, to serve one year:

JOSEPH M. KNAP, New York City.

Directors, to serve three years:

FRANCIS LEE STUART, New York City.
SAMUEL CLARENCE THOMPSON, New York City.
WILLIAM GLYDE WILKINS, Pittsburg, Pa.
ARTHUR NEWELL TALBOT, Urbana, Ill.
WILLIAM MONTGOMERY GARDNER, Memphis, Tenn.
HORACE AUGUSTUS SUMNER, Denver, Colo.

Mr. Benzenberg and Mr. Stearns conducted Mr. Bates, the President-elect, to the chair.

Mr. Bates addressed the meeting briefly.

Adjourned.

February 3d, 1909.—The meeting was called to order at 8.30 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 108 members and 12 guests.

The minutes of the meetings of December 16th, 1908, and January 6th, 1909, were approved as printed in the *Proceedings* for January, 1909.

A paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., entitled "The Purification of Ground-Waters Containing Iron and Manganese", was presented by the author and illustrated with lantern slides and chemical experiments. In making the experiments the

* See page 69.

author was assisted by Mr. D. D. Jackson, Chemist for the New York City Department of Water Supply, Gas and Electricity.

The paper was discussed by Emil Kuichling, M. Am. Soc. C. E., and the author, and written communications from Messrs. W. C. Lounsbury, and Arthur L. Terry, Jr., were read by the Secretary.

The Secretary announced the election of the following candidates by the Board of Direction on February 2d, 1909:

As Members.

Earl Ivan Brown, Wilmington, N. C.
George Mairs Bull, Denver, Colo.
John Angell Fox, Cincinnati, Ohio.
Walter Hayden Graves, Spokane, Wash.
James Charles Haugh, New Orleans, La.
George Pierce Howell, Manila, Philippine Islands.
Richard Warren Knight, Pittsburg, Pa.
William Walter Marr, Chicago, Ill.
John William Moore, Rushville, Ind.
Daniel William Murphy, Klamath Falls, Ore.
George Nelson Randle, Sacramento, Cal.
Orville Hickman Turner, Raton, N. Mex.
Meriwether Lewis Walker, Manila, Philippine Islands.

As Associate Members.

Albert Cornelius Arend, Omaha, Nebr.
McCrea Parker Blair, Saint Boniface, Man., Canada.
Robert Lambert Fowler, Perth Amboy, N. J.
Kenneth Crothers Grant, Harrisburg, Pa.
James Robert Hardesty, Pottsville, Pa.
Lauren Augustus Pettebone, Johnsonville, N. Y.
Edmund Phipps Sanger, Mt. Vernon, N. Y.
Leroy Tallman, Portsmouth, R. I.
Granville Lewis Taylor, Wilkinsburg, Pa.
Walter Robert Wheaton, Washington, D. C.
Charles Newton Young, San Francisco, Cal.

As Associate.

Albert Henry Bromley, Jr., Philadelphia, Pa.

As Juniors.

Josiah Richardson Brooks, Pt. Pleasant, W. Va.
Irvin Sutton Grindrod, Philadelphia, Pa.
Frank Alden Russell, Peabody, Kans.
Robert John Schmid, Spokane, Wash.
Charles Randolph Simpson, Warren, Pa.
Joseph Smith, New York City.
William Rogers Tyler, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on February 2d, 1909:

From Associate Member to Member.

Herbert Clarendon Alden, New York City.
Romeo Thompson Betts, New York City.
Ferdinand Finney Harrington, Norfolk, Va.
Gavin Nelson Houston, Denver, Colo.
Elmer Gove Manahan, New York City.
Ralph Eells Newton, Milwaukee, Wis.
Frederick Celestine Schubert, Portland, Ore.

From Junior to Member.

Erle Leroy Veuve, Los Angeles, Cal.

From Junior to Associate Member.

Arthur Merrick Newberry Blamphin, New Orleans, La.
Walter Gottlieb Federlein, Mogollon, N. Mex.
Diedrich William Krellwitz, West New York, N. J.
Fulton Pace, Guayama, Porto Rico.
Jason Paige, Chicago, Ill.
Albert Orange Smith, Port Jefferson, N. Y.

From Junior to Associate.

George Merrick Herron, Palo Alto, Cal.

The Secretary announced the election of the following candidates by the Board of Direction on September 1st, 1908:

As Juniors.

John Charles Prichard, Columbia, Mo.
Harold S. Williams, Caldwell, Idaho.

The Secretary announced the following deaths:

George Adgate, elected Member April 1st, 1896; died January 25th, 1909.

William Creelman Ambrose, elected Member, April 4th, 1888; died January 3d, 1909.

Daniel Dawson Carothers, elected Member, April 4th, 1894; died January 7th, 1909.

William Price Craighill, elected Member, October 7th, 1885; Honorary Member, March 23d, 1896; Director of the Society during the years 1892-93; President, 1894; died January 18th, 1909.

Baron Emile Theodore Quinette de Rochemont, elected Member, January 3d, 1894; died December 8th, 1908.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

January 20th, 1909.—The Board met, as required by the Constitution, at the House of the Society during the Annual Meeting, January 20th, 1909, at 12.30 P. M., President Bates in the chair, Chas. Warren Hunt, Secretary, and present, also, Messrs. Andrews, Benzenberg, Brackett, Christie, Churchill, Harrison, Hazen, Hodgdon, Knap, Macdonald, Pegram, Schneider, Stearns, Swensson, Talbot, Tillson, Wilkins, and Williams.

The President announced the first business of the Board to be the election of a Secretary.

Mr. Hunt retired.

Chas. Warren Hunt was nominated for Secretary, and every member present voted for his election. The President declared Mr. Hunt unanimously elected Secretary.

Mr. Hunt returned and resumed his duties.

The following Standing Committees of the Board were appointed:

Finance Committee: George H. Pegram, S. C. Thompson, Allen Hazen, W. G. Wilkins, M. T. Endicott.

Publication Committee: Charles L. Harrison, Francis Lee Stuart, George F. Swain, Emil Swensson, Horace Andrews.

Library Committee: George W. Kittredge, James Christie, F. W. Hodgdon, Dexter Brackett, Chas. Warren Hunt.

A Committee on Membership was also appointed.

Adjourned.

February 2d, 1909.—Vice-President Pegram in the chair; Chas. Warren Hunt, Secretary, and present also Messrs. Andrews, Hodgdon, Kittredge, Knap, Noble, Schneider, Stuart, Thompson, and Tillson.

Ballots for Membership were canvassed, resulting in the election of 13 Members, 11 Associate Members, 1 Associate, and 7 Juniors, the transfer of 7 Associate Members to the grade of Member, 1 Junior to the grade of Member, 6 Juniors to the grade of Associate Member, and 1 Junior to the grade of Associate.

The following Committees were appointed to take charge of the next Annual Convention:

Committee of the Board of Direction: Messrs. Hodgdon, Tillson, and Hunt.

Local Committee: Messrs. H. W. Hayes, A. W. Dean, S. E. Tinkham, H. D. Woods, J. F. Stevens, Geo. A. Kimball, and J. W. Ellis.

The resolution adopted by the Annual Meeting and relating to the method of the appointment of the Nominating Committee was considered.

Applications were considered, and other routine business transacted.

Adjourned.

REPORT IN FULL OF THE FIFTY-SIXTH ANNUAL MEETING, JANUARY 20th and 21st, 1909.

Wednesday, January 20th, 1909.—President Macdonald in the Chair; Chas. Warren Hunt, Secretary; and present, also, about 250 members.

Meeting called to Order.

The President.—The meeting will come to order. According to the Constitution, the ballot for officers will close at twelve o'clock noon. The polls will be left open until that time. The tellers, in the meantime, will proceed with the canvass, in order to be able to make a report before the close of the meeting. I will announce the tellers later, and we will hear the report of the Board of Direction.

Report of the Board of Direction.

The Secretary read the Report of the Board of Direction.*

Tellers appointed.

The President.—The following gentlemen are requested to serve as tellers, H. E. Cantwell, S. C. Hulse, and George W. Parsons.

The Secretary.—If any of these gentlemen are present, I wish to say that arrangements have been made for the canvass of the ballots on the third floor, and a number of assistants in the office will aid in opening the ballots (of which there are about 1 100) so that they can get through quickly.

Shall I read the report of the Secretary, sir?

The President.—Yes.

Report of the Secretary.

The Secretary read his Report of Receipts and Disbursements.†

The Secretary.—This report shows that notwithstanding the fact that we have paid $10 000 on the mortgage there is a balance of some $12 000 or $13 000 more than a year ago. The balance sheet, Mr. President, which accompanies this report, and is in the hands of all members, seems to indicate that the available assets of the Society are somewhat over $300 000, about $370 000, if the value of the books in the library is included.

The President.—Have you the Treasurer's Report?

Report of the Treasurer.

The Secretary read the Report of the Treasurer.‡

The President.—If there is no objection, the reports read by the Secretary will be accepted and passed to file.

The Secretary.—The following report was received by the Board of Direction from the Committee to Recommend the Award of Prizes.

Award of Prizes.

"The Board of Direction,
"American Society of Civil Engineers,
"220 West 57th Street, New York City, N. Y.

"Gentlemen:

"The undersigned, appointed by letter of the Secretary of the Society dated June 15, 1908, members of a committee to recommend the award of prizes for papers presented to the Society for the year

* See *Proceedings*, Vol. XXXV, p. 9 (January, 1909).
† See *Proceedings*, Vol. XXXV, p. 14 (January, 1909).
‡ See *Proceedings*, Vol. XXXV, p. 17 (January, 1909).

ending with the month of July, 1908, have the honor to report that having given the matter due consideration we unanimously recommend

> "That the Collingwood prize for Juniors be awarded to Article No. 1066, 'Reinforced Concrete Towers' by D. W. Krellwitz;
>
> "That the Rowland Prize be awarded to Paper No. 1067, 'Water Purification at St. Louis, Mo.' by Edward E. Wall;
>
> That the Norman Medal be awarded to No. 1071, 'Movable Bridges' by C. C. Schneider.

"Paper No. 1056, 'Effect of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions' contains articles of great interest and merit prepared by sub-committees of the San Francisco Association of Members of the American Society of Civil Engineers, and we are of opinion that it would be appropriate and graceful for the Society to take official notice of this paper. Although this matter lies outside the duty assigned to us, we invite the attention of the Board of Direction to it, for such action as the Board may think appropriate.

"JOHN W. ALVORD (Chicago, Ill.),
"C. H. MCKINSTRY (San Francisco, Cal.),
"GEORGE G. ANDERSON (Denver, Colo.)."

THE SECRETARY.—The Board of Direction has awarded the prizes to the gentlemen named, as recommended by the Committee, and has also, in accordance with the recommendation of the Committee, expressed to the members of the San Francisco Association, the appreciation of the Board and of the Society of the interesting and valuable papers presented.

Time and Place of Annual Convention.

THE PRESIDENT.—The report as to the time and place of the next Convention, Mr. Secretary?

THE SECRETARY.—The Board of Direction has decided, Mr. President, that the next Annual Convention of the Society shall be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to 9th, 1909, inclusive.

I might say that that hotel will accommodate at least 500, and we are to have the exclusive use of it. In case there should be 600 or 700 there is another hotel, under the same management, near by, which can take care of the overflow, and if the attendance should run up to 800 or 900, the Fabian House is only about half a mile away, so there appears to be no doubt that all the members who want to go to this Convention and their families can be accommodated.

Special Committee on Columns and Struts.

I have to announce, Mr. President, that the Special Committee authorized by vote of the Society to consider and report upon "The Design, Ultimate Strength and Safe Working Values of Steel Columns and Struts" has been appointed by the Board and has organized as follows: A. L. Bowman, Chairman, Lewis D. Rights, Secretary, together with the following members making up the committee, Alfred P. Boller, Emil Gerber, Charles F. Loweth, Ralph Modjeski, Frank C.

Special Committee on Columns and Struts (continued).

Osborn, George H. Pegram, George F. Swain, Emil Swensson, and Joseph R. Worcester.

A. L. Bowman, M. Am. Soc. C. E.—Mr. President, the Special Committee appointed to investigate and report upon "The Design, Ultimate Strength and Safe Working Values of Steel Columns and Struts" reports as follows to the Members of the American Society of Civil Engineers in Annual Meeting:

Report of Special Committee on Columns and Struts.

To the Members of the Am. Soc. C. E.
in Annual Meeting:

The Special Committee authorized by vote of this Society "To consider and report upon the design, ultimate strength, and safe working values of steel columns and struts," having recently organized, desires to emphasize the national importance and necessity of compression tests, and recommends that the following resolution be adopted by this meeting:

Whereas, no full size tests on large compression members have been made, there being no testing machine of sufficient magnitude for the purpose, and

Whereas, the necessity for such tests has been fully established, and the results obtained from them would add greatly to engineering knowledge and be of material benefit to the industries of this country, and

Whereas, it is the sense of the American Society of Civil Engineers in Annual Meeting assembled, that the building of a machine capable of testing to destruction full size compression members of large dimensions, and of accurately recording results, is beyond the means of private interests, and can best be undertaken by the United States Government.

Resolved, that the United States Government be hereby requested to make a sufficient appropriation for and proceed with the construction of a testing machine which will accomplish the desired results.

Resolved, that the Secretary be directed to forward copies of this resolution to the President of the United States, the Vice-President and the Speaker of the House of Representatives.

For the Committee,

A. L. Bowman,
Chairman.

Charles B. Ball, M. Am. Soc. C. E.—This would seem to be a matter which the Federal Government only can handle properly, and I am sure that such a mild recommendation as this Committee proposes ought to be generally acceptable to this meeting. I therefore move that the report of the Committee—I presume it is merely a preliminary report—be accepted, and that the resolution as recommended be passed.

The motion was duly seconded.

The President.—Gentlemen, you have heard the motion. It has been duly seconded. Is there any discussion upon the subject? All

in favor of that motion will signify it by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—Mr. President, the next business is the appointment by this meeting of seven members of the Society as members of the Nominating Committee, one from each of the seven geographical districts into which the Society is divided for the purpose of the Nominating Committee. Nominating Committee.

In District No. 1 the total number of suggestions received is 233. Mr. Frederick W. Gardiner has received 148 votes; Mr. George S. Rice, 20; *Mr. J. G. Gardiner, 4; Mr. Arthur S. Tuttle, 3; Mr. Henry W. Hodge, 3; and Messrs. James H. Edwards, Nelson P. Lewis, Theodor S. Oxholm and George C. Whipple have received 2 votes each.

The following have received one vote each:

W. H. BURR,
L. BUSH,
C. E. CARPENTER,
A. B. CORTHELL,
ALBERT S. CRANE,
RICHARD T. DANA,
H. F. DUNHAM,
LORING N. FARNUM,
GEORGE B. FRANCIS,
ALBERT I. FRYE,
E. P. GOODRICH,
CHARLES S. GOWEN,
CHARLES H. GRAHAM,
L. E. GREGORY,
C. L. HARRISON,
G. A. HARWOOD,
W. J. HASKINS,
A. L. A. HIMMELWRIGHT,
ALLEN HAZEN,
CARY T. HUTCHINSON,
C. M. INGERSOLL, JR.,
GORGE A. JUST,
*LEONARD METCALF,
EDWARD P. NORTH,
H. L. OESTREICH,
C. J. PARKER,
J. R. SAVAGE,
M. R. SHERRERD,
J. WALDO SMITH,
JOSEPH STRACHAN,
EDWIN THACHER,
JOHN G. VAN HORNE,
JOHN C. WAIT,
*D. A. WATT,
CHARLES E. WELLS,
SAMUEL WHINERY,
W. J. WILGUS.

It was duly moved and seconded that Mr. Frederick W. Gardiner be elected a member of the Nominating Committee from District No. 1.

THE PRESIDENT.—It is moved and seconded that Mr. Gardiner be made a member of the Nominating Committee from District No. 1. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—From District No. 2 the total number of votes received is 54. Mr. Richard A. Hale has received 13 votes; Mr. J. R. Worcester, 12; Mr. Harrison P. Eddy, 5; and Messrs. Charles A. Allen,

* Of the above, J. G. Gardiner is not a member of the Society, and Messrs. Leonard Metcalt and D. A. Watt do not reside in District No. 1.

Nominating Committee (continued).

Desmond FitzGerald, Hector J. Hughes, Hiram A. Miller, Albert L. Scott and *George F. Swain have received 2 votes each.

The following have received one vote each:

H. K. Barrows,	George A. Kimball,
George A. Carpenter,	Henry Manley,
Fayette S. Curtis,	Leonard Metcalf,
J. R. Freeman,	William H. Moore,
Edward Gagel,	O. Perry Sarle,
*C. M. Ingersoll, Jr.,	*F. P. Stearns.

Frederic P. Stearns, Past-President, Am. Soc. C. E.—I move that Mr. Richard A. Hale be elected a member of the Nominating Committee from District No. 2.

The motion was duly seconded.

The President.—You have heard the motion, gentlemen. All in favor signify by saying "aye"; contrary, "no." It is a vote.

Joseph M. Knap, Treasurer, Am. Soc. C. E.—I move that hereafter only the first three names be read. It is not worth while perhaps to read the names of those who get only one vote, and it would expedite things if only the first three names were read.

The President.—If there is no objection that seems to be a very good suggestion for the Secretary to observe.

The Secretary.—From District No. 3 the total number of votes received was 111. Mr. M. G. Barnes has 30 votes; Mr. Edward B. Codwise, 27; and Mr. E. E. Haskell, 7.

Carlton E. Davis, M. Am. Soc. C. E.—Mr. President, I move that Mr. Codwise be appointed a member of the Nominating Committee; though he has not received as many votes as Mr. Barnes, yet I think that his appointment will serve better to represent the District. There is one member of the Nominating Committee from District No. 3 who has still a year to serve, Mr. David A. Watt, who is a member of the State Engineer's Office at Albany. Mr. Barnes is also a member of the State Engineer's Office at Albany, so that it seems to me that the election of Mr. Codwise will be of more universal service to the Society, and not concentrate the entire representation in the State Engineer's Office, at Albany.

The motion was duly seconded.

Joseph Ripley, M. Am. Soc. C. E.—I move to amend that motion by substituting the name of M. G. Barnes.

The motion was duly seconded.

The President.—The vote on the amendment will come first in the natural course.

The Secretary.—Would it not be simpler, Mr. President, to withdraw all motions and to vote on each of the names.

* Of the above, Messrs. George F. Swain and F. P. Stearns are not eligible, being members of the Board of Direction; and C. M. Ingersoll, Jr., does not reside in District No. 2.

Mr. Webster.—I make a motion to that effect.

The President.—You have heard the motion that we take a vote on the two names mentioned from the Third District. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

Mr. Knap.—I move that it be a rising vote.

The President.—All in favor of Mr. Barnes's election will please rise.

A Member.—Will you kindly state what the motion is?

The President.—Mr. Barnes's name will now be voted upon. All in favor of his appointment from the Third District will please rise.

The Secretary.—Twenty.

The President.—All in favor of Mr. Codwise will please rise.

The Secretary.—Sixty-seven.

The President.—I declare Mr. Codwise duly elected from District No. 3.

The Secretary.—From District No. 4 the total number of votes received is 110. Mr. Frank Sutton has 40 votes; Mr. Richard L. Humphrey, 19; Mr. Calvin W. Hendrick, 4.

Mr. Davison.—I move that Mr. Frank Sutton be elected as member from District No. 4.

The motion was duly seconded.

The President.—You have heard the motion. All in favor signify by saying "aye"; contrary, "no." It is a vote.

The Secretary.—From District No. 5 there are 126 votes, of which Mr. H. E. Riggs has 58; Mr. O. W. Childs, 25; and Mr. Archibald S. Baldwin, 3.

It was duly moved and seconded that Mr. Riggs be appointed a member of the Nominating Committee from District No. 5.

The President.—All in favor of that say "aye"; contrary, "no." It is a vote.

The Secretary.—From District No. 6 the total number of votes received is 79, of which Mr. Charles C. Wentworth has received 9; Mr. J. F. Coleman, 5; Mr. Hunter McDonald, 4. In this case I have to read the fourth name because Mr. Frank M. Kerr has also received 4.

C. S. Churchill, M. Am. Soc. C. E.—I nominate Mr. Charles C. Wentworth to be a member of the Nominating Committee from District No. 6.

The motion was duly seconded.

The President.—It is moved and seconded that Mr. Wentworth be elected a member of the Nominating Committee from District No. 6. All in favor will say "aye"; contrary, "no." It is a vote.

The Secretary.—From District No. 7 the total vote is 108; of which Mr. C. E. Fowler has received 10; Mr. H. J. Burt, 7; Mr. Edwin Duryea, Jr., 7; and Mr. R. H. Thomson, 7.

George H. Pegram, M. Am. Soc. C. E.—I move that Mr. Fowler

Nominating Committee (continued).

be elected as a member of the Nominating Committee from District No. 7.

The motion was duly seconded.

THE PRESIDENT.—It is moved and seconded that Mr. Fowler be elected a member of the Nominating Committee from District No. 7. All in favor of that signify by saying "aye"; contrary "no." It is a vote.

Report of Special Committee on Status of the Metric System.

THE SECRETARY.—The next order of business is the report of the Committee on the Status of the Metric System. This report was published in the *Proceedings* for October, 1908, and copies, in pamphlet form, are available for distribution to those in attendance at this meeting.

I also have a discussion and criticism of this report from Mr. Henry R. Towne, who is not a member of the Society; but if I might venture to make a suggestion, this Committee has made what is intended to be a final report, and wishes to be discharged. Now, whether this report should be printed without any discussion or not is a matter for this meeting to determine, but it does seem to me that this report might be treated as a paper of the Society, and published with the discussions to which it gives rise, because there is a good deal of difference of opinion as to the availability of the metric system, and it might lead to something.

THE PRESIDENT.—The correct course, gentlemen, would be to move to discharge the committee with thanks, and print the report.

It was moved and seconded that the Committee be discharged with thanks, and that the report be printed.

THE PRESIDENT.—You have heard the motion, duly seconded, that this Committee be discharged, with the thanks of the Society, and that its report be printed. All in favor of that signify by saying "aye"; contrary, "no." It is a vote.

Report of Special Committee on Concrete and Reinforced Concrete.

THE SECRETARY.—The next business in order is the Report of the Special Committee on Concrete and Reinforced Concrete.* This report, which is signed by six members of the Committee, and a minority report signed by the other four members, is available in pamphlet form for distribution to those in attendance, and perhaps need not be read. Mr. Humphrey, the Secretary of the Committee, is here, I believe.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—Mr. President, the report cited is the report agreed upon by a majority of the Special Committee on Concrete and Reinforced Concrete, and is also the report agreed upon by the Joint Committee appointed by the American Society for Testing Materials, the American Railway Engineering and Maintenance of Way Association, and the Association of American

*The majority and minority reports of this Committee are printed on pages 85 to 120 of this number of *Proceedings*.

Portland Cement Manufacturers, with which your Special Committee co-operated, and it is printed for the purpose of getting discussion on it. The Committee is not able to present a final report at this time.

The report itself is presented for the reason that the majority of the members of the committee believe that the information which the committee has acquired and the views expressed would be of value and interest to the members of the Society. The committee desires that the report be printed and circulated among the members and that the committee be continued.

A MEMBER.—I second that motion.

THE SECRETARY.—I call attention to the fact that there is a Minority Report, which is not very long, and ask whether it is the pleasure of the meeting that it be read.

THE PRESIDENT.—I understand that the gentleman's motion is that the Majority and Minority reports be printed and distributed among the members, leading up to a discussion at some future meeting, and that the Committee be continued.

THE SECRETARY.—Might I venture to suggest, Mr. President, that it might be wise to insert in that motion the date on which the discussion should be taken up, and also that the next Annual Convention would be a very good time.

A MEMBER.—I accept that.

The motion was duly seconded.

THE PRESIDENT.—You have heard that motion. All in favor say "aye"; contrary, "no." The ayes appear to have it; the ayes have it. It is a vote.

Report of Special Committee on Uniform Tests of Cement.

THE SECRETARY.—The Report of the Special Committee on Uniform Tests of Cement is here in pamphlet form, and I will simply read the last few lines of it.*

A MEMBER.—I move that the Report be received and the Committee continued.

Motion duly seconded.

THE PRESIDENT.—You have heard the motion. All in favor of it signify by saying "aye"; contrary, "no." It is a vote.

Report of Special Committee on Rail Sections.

THE SECRETARY.—I have the following brief report from the Special Committee on Rail Sections:

DECEMBER 18TH, 1908.

AMERICAN SOCIETY OF CIVIL ENGINEERS,

GENTLEMEN:—Your Committee respectfully reports that since its report of December 6th, 1907, it has continued its investigations, keeping in touch with the progress made by railroads, and the other engineering associations of the country, as well as experiments and tests made by the United States Government at the Watertown Arsenal, Watertown, Mass.

The progress made on this subject has been slow, but quite im-

* This report is printed on pages 71 to 84 of this number of *Proceedings*.

Report of Special Committee on Rail Sections (continued).

portant. The information gathered is quite voluminous—so much so that it is hardly necessary to burden this Society with much of it, as there is no body of men, in railroad circles or in the different societies, which is ready to-day to make a recommendation for a rail section to be adopted by all the railroads. Your Committee is therefore not yet prepared to submit respective sections for the different weights of rail. The cardinal principles submitted in your Committee's Report of December 6th, 1907, however, seem to have been generally accepted by all professional men and railroads making a study of the subject, and so far as sctions of rail are being made we believe they are accepted as good and being adhered to, and it is gratifying to your Committee to be able so to report.

We may add that since our last Report the American Railway Association has placed its work in the hands of a Committee of Operating Engineers of the American Railway Engineering and Maintenance of Way Association of Chicago, and asked that Committee to report on a rail section, this report to be returned to the American Railway Association to be considered for adoption. Three members of your Committee are members of the Committee of the American Railway Engineering and Maintenance of Way Association, and a greater number of them are members of the American Railway Association. Also, the American Society for Testing Materials has a Committee, several of the members of which are members of your Committee, so that the work of the several Committees is interwoven to such an extent that they are working in harmony.

The American Society for Testing Materials, since our report, has made some important changes in its specifications, bringing them more into harmony with the specifications proposed by this Committee.

We also desire to call attention to the increased use of open-hearth steel rails, and of the greater facilities we will have for obtaining open-hearth steel rails in the future.

These Associations have also joined in collecting information as to rail failures, and have adopted a uniform blank for reporting rail failures, which will give information much more in detail and more accurate than the information that has generally been collected by the railroads hitherto.

They have also joined in having tests made in ingots, blooms, rails and rail joints, under the supervision of the United States Arsenal at Watertown, Mass. All this work will add very much to the data available in the study of the making of rails at the mills, and the wear and failure of them on the railroads; and, considering the situation as it stands, your Committee would deem it wise to make this a Report of Progress, and ask to be continued.

We respectfully remain,

JOSEPH T. RICHARDS, *Chairman.*
ROBERT W. HUNT, *Secretary.*
C. W. BUCHHOLZ,
E. C. CARTER,
S. M. FELTON,
JOHN D. ISAACS,
RICHARD MONTFORT,
H. G. PROUT,
PERCIVAL ROBERTS, JR.,
GEORGE E. THACKRAY,
EDMUND K. TURNER,
WILLIAM R. WEBSTER.

THE PRESIDENT.—What is your pleasure in regard to this report?

It was duly moved and seconded that the Report be received and the Committee continued.

THE PRESIDENT.—You have heard the motion that the Report be received and the Committee continued. All in favor signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—Mr. President, from the Chairman of the Special Committee on Engineering Education, Mr. Desmond FitzGerald, I have received the following note: Special Committee on Engineering Education.

"BROOKLINE, MASS.,
16 JANUARY, 1909.

"MY DEAR MR. HUNT

"As a partial progress report of the 'Education Committee' I send you these few lines. During last year a Joint Committee on Engineering Education has been formed consisting of the following members:

"Representing Com. of Am. Soc. C. E.	Desmond FitzGerald, Chn.
	B. M. Harrod
"Inst. Mining Engineers	Dr. Henry M. Howe
	John Hays Hammond
"Am. Chemical Socy	Prof. H. P. Talbot
	Dr. Clifford Richardson
"Inst. Electrical Engineers	Dr. Samuel Sheldon
	C. F. Scott
"Am. Socy. Mech. Engrs	Prof. Alex. C. Humphreys
	F. W. Taylor
"Society for Eng. Education	Prof. Dugald C. Jackson
	Prof. C. L. Crandall
	Dean James M. White
"Genl. Ed. Board	E. Benj. Andrews
"Carnegie Foundation	Henry S. Prichett

"This Committee has organized and have done me the honor to elect me Chairman.

"It was determined to begin the work of the Committee by making a study of the present methods of teaching Engineering and this work is now in progress.

"Very truly yours,
"DESMOND FITZGERALD."

THE PRESIDENT.—The next business is the proposed amendment to the Constitution. This amendment was offered by Mr. Clemens Herschel and other members of the Society, was considered by the Business Meeting of the Annual Convention in Denver, June 23d, 1908, and was in the following form: Proposed Amendment to the Constitution.

Proposed Amendment to the Constitution (continued).

Amend Section 4 of Article III as follows: Instead of the word "twenty" in the fifth line thereof, insert the words "one-fifth"; and after the word "votes" in the same line, insert the words: "of all the votes cast". So that the new sentence shall read: "One-fifth or more negative votes of all the votes cast shall exclude from election."

This was amended by the Business Meeting of the Annual Convention to read as follows:

"Negative votes equal to one per cent., or to the whole number nearest to one per cent., of the total corporate membership at the time of voting shall exclude from election."

The amendment as amended was referred to a special committee to report at this Annual Meeting. The committee so appointed consists of Messrs. George H. Pegram, John W. Ellis and Henry R. Leonard.

Mr. Pegram.—I hand you a report.

The Secretary.—Do you want me to read it?

Mr. Pegram.—If you please.

The Secretary.—The report is as follows:

January 16th, 1909.

Report of Committee on Proposed Amendment to the Constitution.

To the American Society of Civil Engineers:

Your Committee, appointed pursuant to a resolution of the Business Meeting at Denver, June 23d, 1908, to consider an amendment to the Constitution proposed by Mr. Clemens Herschel and others, relating to the election of members, begs to report as follows:

The proposed amendment is:

"Amend Section 4 of Article III as follows:

"Instead of the word 'twenty' in the 5th line thereof insert the words 'One-fifth'; and after the word 'votes' in the same line, insert the words: 'of all the votes cast.' So that the new sentence shall read:

" 'One-fifth or more negative votes of all the votes cast shall exclude from election.' "

This was amended by the Denver Meeting by substituting, "One per cent. of the corporate membership", for, "One-fifth of all the votes cast".

The proposed amendment, together with two others relating to the same subject, was considered by a Committee which reported to the Denver Meeting at which the matter was discussed, and the majority report of the Committee together with the expressions of the views of the other members were printed and sent out with the letter-ballots on the amendment placing the election of members in the Board of Direction, which was adopted.

The merits of the amendments were so thoroughly considered in the report referred to, and the question has been so generally discussed, and apparently settled by the adoption of the amendment placing the election of members in the Board of Direction, that the duty of your Committee would seem to be to advise how the proposed amendment can be properly disposed of.

The purpose of this amendment was to correct a defect in the Constitution, which has been corrected, and it might appear that it is therefore void; but the Constitution requires that proposed amendments shall be voted upon by letter-ballot after having been con-

sidered by a general meeting which has power only to amend it in a manner pertinent to the original amendment, and if not amended it shall be voted upon by letter-ballot in the original form.

There would, therefore, seem to be no power to declare the proposed amendment void, and it would be a dangerous precedent to assume it so, because there might be proposed amendments which, while anticipated in part by amendments adopted since their presentation, would still have desirable features not affected by such action.

On the other hand, the discussions and action in this matter would indicate that the adoption of this proposed amendment is not desired or, in fact, advisable until the Constitution as amended October 7th, 1908, has had a period of trial.

Your Committee, therefore, recommends that the proposed amendment be re-worded by the Secretary to apply to the Constitution as it now exists, and sent out to letter-ballot with an accompanying statement that it must be voted upon by letter-ballot to meet the requirements of the Constitution; but that in view of the adoption of an amendment since its presentation placing the election of members in the hands of the Board of Direction, it is the opinion of this Committee that it should be defeated.

Respectfully submitted,

Geo. H. Pegram,
John W. Ellis,
H. R. Leonard,
Committee.

J. A. Bensel, M. Am. Soc. C. E.—I move that the Report of the Committee be received by the Society and its recommendations adopted.

The President.—You have heard the motion, gentlemen. Is the motion seconded?

The motion was duly seconded.

The President.—All in favor of that motion signify by saying "aye"; contrary, "no." It is a vote.

Announcements.

The Secretary.—Mr. President, I take this opportunity of making an announcement in regard to the programme. This announcement has been made by placards, which are posted throughout the House, but somebody may not have seen them.

The excursion this afternoon is by automobile. The number of people who said they would come to this meeting—some 1 100—make it somewhat uncertain as to how many can be taken on that trip, and the limit of possibility in that direction is about 500 persons.

So it was determined to issue tickets, and to ask the resident members, who can at all times see the bridges, to wait until the non-residents have secured the tickets they need before applying for their own.

In addition to this, through the courtesy of the Commissioner of the Department of Charities, a steamer which will hold at least 250 people, will be in waiting at the foot of East Sixtieth Street, and will leave there at 3.45 this afternoon, sailing down the East River, and

Announcements (continued). around the Battery, and up to the Chelsea Docks to meet the party that goes in automobiles.

I presume everybody has a copy of the programme of the meeting, and, as far as I know, there has been no change in the times of trains or in anything else. The special train to the Ashokan Reservoir, to-morrow, will stop at Newburg to take on passengers, and will also stop at Kingston. The time of arrival at Kingston is 12.05, and at Newburg about an hour earlier, so that those in that vicinity need not come to New York to take the special train.

I do not think I have anything else to say, Mr. President. I know of no other business at the present moment.

THE PRESIDENT.—If there is any new business to be considered we have ample time, as the ballot does not close until 12 o'clock.

Method of Appointing the Nominating Committee.

SAMUEL WHINERY, M. AM. SOC. C. E.—Mr. President, it has been evident for many years that the method now in vogue for choosing the Nominating Committee of the Society has worked unsatisfactorily. Perhaps no better illustration of that fact could be adduced than our proceedings here to-day. Out of a very large membership, the results, as read by the Secretary to-day, show not only that a very small part of the membership has taken any part whatever in this very important question of selecting the Nominating Committee, and that a similar small part, a mere fraction of the total membership of the Society, had any part in selecting the persons who have been elected here to-day as members of that Nominating Committee.

That has been the case for many years, and it seems to me it is a situation that demands the attention of the Society. I have taken some pains to collect the figures for the past six years, that is, from 1903 to 1908. It appears that the Nominating Committee of this Society, on whom depends the nomination of the officers of the Society—a most important function—that the members of the Nominating Committee have been elected upon the basis of $7\frac{2}{10}\%$ of the membership of the Society. In other words, $7\frac{2}{10}\%$ of the corporate membership of the Society, upon the average in the last five years, have dictated the election of the officers of this Society. It has appeared in that time that in a number of cases, as few as two (I won't say within this period as few as two, but previous to that I remember a case where only two members voted in a District) voted for the person who was officially elected as the member of the Nominating Committee.

It has often occurred that five or less members in this meeting have dictated the member of the Nominating Committee. It appears further—I might say to begin with that the method of electing the Nominating Committee of the Society is probably familiar to you, but the provisions of the Constitution are very short, and it will take but a moment to read them to you.

"At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting to serve for two years; who, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

"The Board of Direction may prescribe the mode of procedure for appointing this Committee.

"The Committee so appointed shall meet at the Annual Convention of the Society, and nominate candidates to fill the offices, named in Article V, so as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors, divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6 and 7."

The procedure has been that about two months before the Annual Meeting of the Society the Secretary sends out printed blanks to the Corporate Members, asking that the blanks shall be returned filled out with the member's suggestion of a name for member of the Nominating Committee. When these are returned the results are collated by the Secretary, as has been done to-day. It will appear that some one person has more votes than another usually. Occasionally two or three will have the same number, but usually one member has a larger number of votes; but be that number two, three, or four or ten, it has been the universal custom, I think, in this Society, with one or two exceptions—one of which occurred to-day—to elect that person as a member of the Nominating Committee who has the largest number of votes.

Now, that is certainly not the basis on which the American Society of Civil Engineers should proceed to elect the officers of the Society, and it seems to me that while the matter has worked fairly well so far, it is high time that the Society should make some change in that respect. There is no criticism to be offered with reference to the *personnel* or the work of past Nominating Committees. We have been very fortunate, I think, in that respect. That good fortune, however, is due to the high character of the men who have happened to be elected to the Nominating Committee—I say "happened", because that is precisely the fact—and to the good sense of the membership, and the absence in the Society of any disposition of selfishness or political action. To that alone has been due the fact that we have done as well as we have.

The facts show that 74 Corporate Members, in 1903, elected a majority of the one-half of the Nominating Committee elected that year. Of course, you understand that the Nominating Committee consists now of 19 persons, 14 of whom are elected. Of the 19, 10 form a majority, and would be able under any circumstances to control the nominations or the actions of the Committee.

Now, in 1903 only 74 Corporate Members effected the election of

Method of Appointing the Nominating Committee (continued).

five members of the Nominating Committee; in the next year 78 Corporate Members dictated the nomination of five members. So that about 75 members in those two years elected a controlling element in the Nominating Committee. In the next year, 1905, only 50 Corporate Members elected five of the seven members elected that year. The next year, 1906, was a more fortunate year, 120 then were required to elect the five members. In the next year, 1907, only 65 Corporate Members elected five members of the Committee.

It appears, therefore, from these figures, that if it were true in this Society, as it is in some others, that selfish methods or personal interests prevailed, or political methods, it would be entirely practicable for a bunch of say half a dozen members of the Society in the course of two years to secure absolute control of the Nominating Committee.

Now, while we have had no bad results from this in the past, and while there is no reason to anticipate that we shall in the future, still it is a condition of affairs that we ought to remedy. It ought not to be possible in the American Society of Civil Engineers for a bunch of less than one hundred men to dictate who shall be the officers of the Society, as is possible now.

It would not be possible for an Annual Meeting to proceed differently from what it does. The members act on the spur of the moment. There is no time for consideration or for conference as to the fitness of this or that person, and the Annual Meeting can only do practically as it has done to-day.

The trouble seems to lie with the apathy or the neglect of the members to reply to the circular letter of the Secretary. But there must be some cause antecedent to this. Surely on a matter of this importance more than 200 or 300 members will take sufficient interest in the welfare of this Society to fill out and sign a simple blank. The fact is, however, that these circular letters go to the members, many of whom are busy. They lay them aside. There is nothing to call their special attention to the matter or to the importance of it. In a great many cases they are never filled out, or only returned in very few cases. Is there in the districts any consideration, any means, any method or attempt to concentrate the views of the membership upon some one person, who would form a suitable member of the Nominating Committee? It seems to me that this is a defect in the present methods of the Society, and an important matter that we should take some means to remedy. That remedy, according to the Constitution, is placed in the hands of the Board of Direction.

I, therefore, Mr. President, offer the following preamble and resolution:

"*Whereas:* The present method of appointing the Nominating Committee of the Society has, in practice, proved to be unrepresentative and unsatisfactory, and

"*Whereas:* Section 2 of Article VII of the Constitution provides that the Board of Direction may prescribe the mode of procedure for appointing the Nominating Committee, therefore be it

"*Resolved:* That the Board of Direction be requested to give further consideration to the whole subject of the method of nominating officers for the Society; and particularly to the present mode of procedure in ascertaining and recording an expression of the corporate membership with regard to the choice of members of the Nominating Committee in the several geographical districts; and to report its conclusions and recommendations to the next Annual Convention of the Society."

Mr. President, I move the adoption of that resolution.

The motion was duly seconded.

THE PRESIDENT.—You have heard the motion, gentlemen. All in favor of it signify by saying "aye"; contrary, "no." It is a vote.

THE SECRETARY.—The Tellers are not ready to report, but they are very nearly ready, and I have no doubt from what I have seen that they will be ready about 12 o'clock.

THE PRESIDENT.—If there is no further business, it would be desirable to take a recess until the Tellers report.

GARDNER S. WILLIAMS, M. AM. SOC. C. E.—I move that we take a recess until five minutes of twelve.

The motion was duly seconded.

THE PRESIDENT.—You have heard the motion. All in favor of that signify it by saying "aye." Recess.

THE SECRETARY.—Before you rise, I would like to suggest the probability that we will have to serve luncheon to a number of you in this room, so that it will not be necessary for you to go down stairs.

After recess:

THE PRESIDENT.—The report of the Tellers will now be read. Ballot for Officers.

THE SECRETARY.—Mr. President, the Tellers have had a very hard struggle, and they have not been able to get their report in very good shape, but the results are here. The Tellers, Messrs. G. W. Parsons, S. C. Hulse, and Herbert E. Cantwell, report the total number of votes received and counted as 1 083.

For President:	
ONWARD BATES	1 072
Scattering	5
For Vice-Presidents:	
GEORGE H. PEGRAM	1 063
EMIL SWENSSON	1 043
Scattering	26
For Treasurer:	
JOSEPH M. KNAP	1 075
Scattering	1

Ballot for Officers (continued).

For Directors:

F. L. Stuart	1 032
S. C. Thompson	1 046
W. G. Wilkins	1 041
A. N. Talbot	1 038
W. M. Gardner	1 044
H. A. Sumner	1 042
Scattering	53

Officers Elected.

The President.—In accordance with the vote which you have just heard read there have been elected for President, Onward Bates; Vice-Presidents, George H. Pegram and E. Swensson; for Treasurer, Joseph M. Knap; for Directors, F. L. Stuart, S. C. Thompson, W. G. Wilkins, A. N. Talbot, W. M. Gardner, H. A. Sumner; and I declare these gentlemen elected to the offices indicated.

I will ask Mr. Benzenberg and Mr. Noble if they will be good enough to escort the new President to the Chair.

Mr. Benzenberg.—I do not think Mr. Noble is in the room.

The President.—Is Mr. Stearns here?

Mr. Bates was escorted to the Chair by Past-Presidents Benzenberg and Stearns.

The President.—It is my great pleasure, sir, to announce your election as President for the ensuing year, and to present you with this gavel, and to wish you every success in your new position.

Mr. Bates.—Gentlemen, and members of the American Society of Civil Engineers, an engineer should always be prepared to accept promotion to higher and better work. I am very well aware that the American Society of Civil Engineers in electing me its President has conferred on me its highest honor, but I prefer to think of it as an engagement for service. Therefore, I lay the honor on the table during my time of service, and will endeavor to fill the office and meet the trust which has been given to me by my brethren until I complete my term of office and have my name added to the list of honored Past-Presidents. I may then rejoice in the honor, which will be a lasting one.

This is an occasion for few words, and about all that I can add is that I fail to find words which seem to me to express properly my grateful appreciation.

The Secretary.—Mr. President, I think it only remains to announce that a meeting of the Board of Direction will be held as soon as this meeting adjourns, and to ask all members of the Board of Direction if they will not come to the Committee Room, on the first floor, adjoining the Secretary's office at once, in order to get through with the business there is to be done.

Mr. Macdonald.—I move we adjourn.

The motion was duly seconded.

The President.—Those in favor of adjournment will please say "aye"; contrary, "no." Carried.

Adjourned.

REPORTS OF SPECIAL COMMITTEES.

PROGRESS REPORT OF SPECIAL COMMITTEE ON UNIFORM TESTS OF CEMENT.

PRESENTED AT THE ANNUAL MEETING, JANUARY 20TH, 1909.

Your Committee on Uniform Tests of Cement has devoted much time, and given very careful consideration, to the subject. Frequent meetings have been held, and a number of investigations carried on, some of which cannot be finally reported at this time.

On several matters which have been considered, such as the substitution of a natural sand for the standard quartz, and the tests for normal consistency and constancy of volume, the Committee has not reached final conclusions.

While not prepared to submit a final report, the Committee feels that it should present a report of progress in order that the Society may be informed of the results of its investigations and conclusions.

The work of the Committee has been confined entirely to the methods for making tests, without attempting to specify what tests should be made.

In order to do full justice to the subject under consideration, it will be necessary to compile and report the results of the experiments now under way, and your Committee asks, therefore, that it be continued.

SAMPLING.

1.—*Selection of Sample.*—The selection of the sample for testing is a detail that must be left to the discretion of the engineer; the number and the quantity to be taken from each package will depend largely on the importance of the work, the number of tests to be made and the facilities for making them.

2.—The sample shall be a fair average of the contents of the package; it is recommended that, where conditions permit, one barrel in every ten be sampled.

3.—Samples should be passed through a sieve having twenty meshes per linear inch, in order to break up lumps and remove foreign material; this is also a very effective method for mixing them together in order to obtain an average. For determining the characteristics of a shipment of cement, the individual samples may be mixed and the average tested; where time will permit, however, it is recommended that they be tested separately.

4.—*Method of Sampling.*—Cement in barrels should be sampled through a hole made in the center of one of the staves, midway between

the heads, or in the head, by means of an auger or a sampling iron similar to that used by sugar inspectors. If in bags, it should be taken from surface to center.

Chemical Analysis.

5.—*Significance.*—Chemical analysis may render valuable service in the detection of adulteration of cement with considerable amounts of inert material, such as slag or ground limestone. It is of use, also, in determining whether certain constituents, believed to be harmful when in excess of a certain percentage, as magnesia and sulphuric anhydride, are present in inadmissible proportions.

6.—The determination of the principal constituents of cement—silica, alumina, iron oxide and lime—is not conclusive as an indication of quality. Faulty character of cement results more frequently from imperfect preparation of the raw material or defective burning than from incorrect proportions of the constituents. Cement made from very finely-ground material, and thoroughly burned, may contain much more lime than the amount usually present, and still be perfectly sound. On the other hand, cements low in lime may, on account of careless preparation of the raw material, be of dangerous character. Further, the ash of the fuel used in burning may so greatly modify the composition of the product as largely to destroy the significance of the results of analysis.

7.—*Method.*—As a method to be followed for the analysis of cement, that proposed by the Committee on Uniformity in the Analysis of Materials for the Portland Cement Industry, of the New York Section of the Society for Chemical Industry, and published in *Engineering News,* Vol. 50, p. 60, 1903; and in *The Engineering Record,* Vol. 48, p. 49, 1903, is recommended.

Specific Gravity.

8.—*Significance.*—The specific gravity of cement is lowered by adulteration and hydration, but the adulteration must be in considerable quantity to affect the results appreciably.

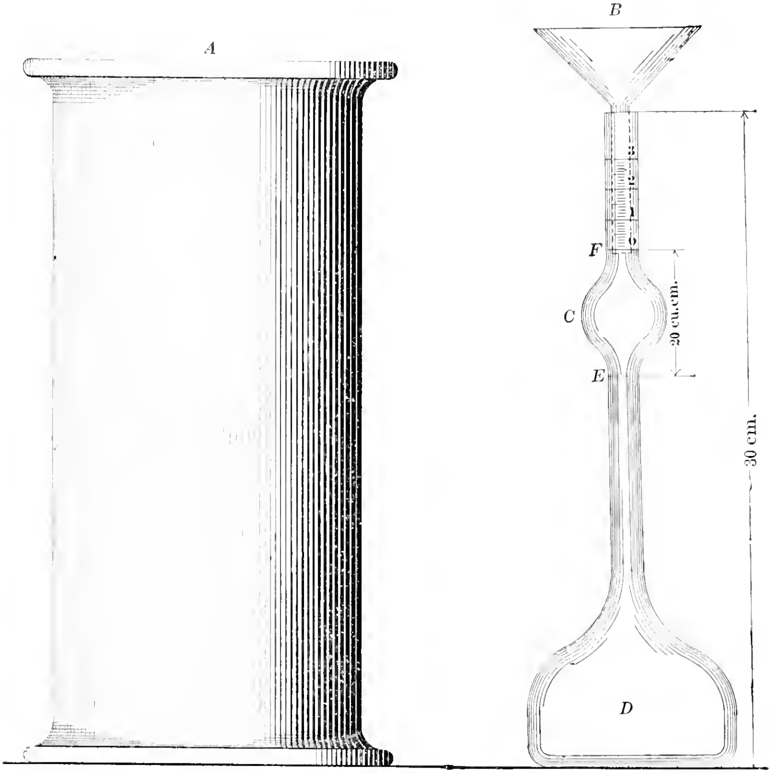
9.—Inasmuch as the differences in specific gravity are usually very small, great care must be exercised in making the determination.

10.—*Apparatus and Method.*—The determination of specific gravity is most conveniently made with Le Châtelier's apparatus. This consists of a flask (*D*), Fig. 1, of 120 cu. cm. (7.32 cu. in.) capacity, the neck of which is about 20 cm. (7.87 in.) long; in the middle of this neck is a bulb (*C*), above and below which are two marks (*F*) and (*E*); the volume between these marks is 20 cu. cm. (1.22 cu. in.). The neck has a diameter of about 9 mm. (0.35 in.), and is graduated into tenths of cubic centimeters above the mark (*F*).

11.—Benzine (62° Baumé naphtha), or kerosene free from water, should be used in making the determination.

12.—The specific gravity can be determined in two ways:

(1) The flask is filled with either of these liquids to the lower mark (*E*), and 64 g. (2.25 oz.) of powder, cooled to the temperature of the liquid, is gradually introduced through the funnel (*B*) [the stem of which extends into the flask to the top of the bulb (*C*)], until the upper mark (*F*) is reached. The difference in weight between the cement remaining and the original quantity (64 g.) is the weight which has displaced 20 cu. cm.



LE CHATELIER'S SPECIFIC GRAVITY APPARATUS.

Fig. 1.

13.—(2) The whole quantity of the powder is introduced, and the level of the liquid rises to some division of the graduated neck. This reading plus 20 cu. cm. is the volume displaced by 64 g. of the powder.

14.—The specific gravity is then obtained from the formula:

$$\text{Specific Gravity} = \frac{\text{Weight of Cement, in grammes,}}{\text{Displaced Volume, in cubic centimeters.}}$$

15.—The flask, during the operation, is kept immersed in water in a jar (*A*), in order to avoid variations in the temperature of the liquid. The results should agree within 0.01. The determination of specific gravity should be made on the cement as received; and, should it fall below 3.10, a second determination should be made on the sample ignited at a low red heat.

16.—A convenient method for cleaning the apparatus is as follows: The flask is inverted over a large vessel, preferably a glass jar, and shaken vertically until the liquid starts to flow freely; it is then held still in a vertical position until empty; the remaining traces of cement can be removed in a similar manner by pouring into the flask a small quantity of clean liquid and repeating the operation.

17.—More accurate determinations may be made with the picnometer.

Fineness.

18.—*Significance.*—It is generally accepted that the coarser particles in cement are practically inert, and it is only the extremely fine powder that possesses adhesive or cementing qualities. The more finely cement is pulverized, all other conditions being the same, the more sand it will carry and produce a mortar of a given strength.

19.—The degree of final pulverization which the cement receives at the place of manufacture is ascertained by measuring the residue retained on certain sieves. Those known as the No. 100 and No. 200 sieves are recommended for this purpose.

20.—*Apparatus.*—The sieves should be circular, about 20 cm. (7.87 in.) in diameter, 6 cm. (2.36 in.) high, and provided with a pan, 5 cm. (1.97 in.) deep, and a cover.

21.—The wire cloth should be of brass wire having the following diameters:

No. 100, 0.0045 in.; No. 200, 0.0024 in.

22.—This cloth should be mounted on the frames without distortion; the mesh should be regular in spacing and be within the following limits:

No. 100, 96 to 100 meshes to the linear inch.
No. 200, 188 to 200 " " " " "

23.—Fifty grammes (1.76 oz.) or 100 g. (3.52 oz.) should be used for the test, and dried at a temperature of 100° cent. (212° Fahr.) prior to sieving.

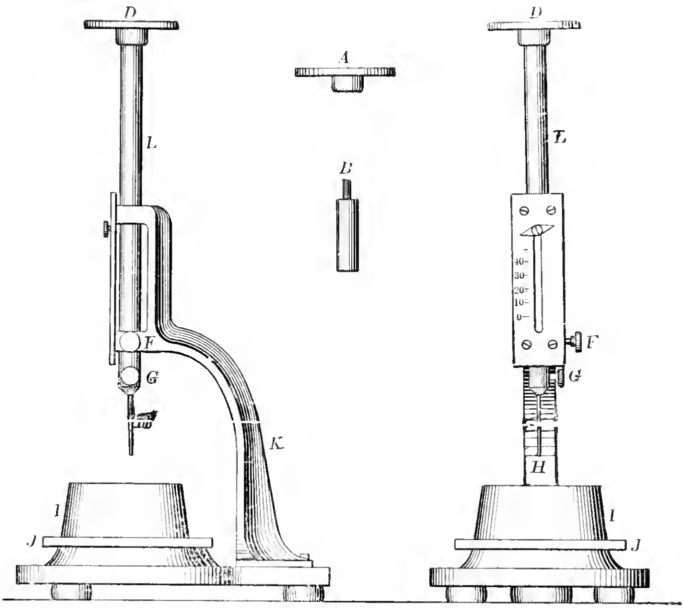
24.—*Method.*—The Committee, after careful investigation, has reached the conclusion that mechanical sieving is not as practicable or efficient as hand work, and therefore recommends the following method:

25.—The thoroughly dried and coarsely screened sample is weighed and placed on the No. 200 sieve, which, with pan and cover attached, is held in one hand in a slightly inclined position, and moved forward and backward, at the same time striking the side gently with the palm

of the other hand, at the rate of about 200 strokes per minute. The operation is continued until not more than one-tenth of 1% passes through after one minute of continuous sieving. The residue is weighed, then placed on the No. 100 sieve and the operation repeated. The work may be expedited by placing in the sieve a small quantity of large steel shot. The results should be reported to the nearest tenth of 1 per cent.

NORMAL CONSISTENCY.

26.—*Significance.*—The use of a proper percentage of water in making the pastes* from which pats, tests of setting, and briquettes are made, is exceedingly important, and affects vitally the results obtained.



VICAT NEEDLE.
FIG. 2.

27.—The determination consists in measuring the amount of water required to reduce the cement to a given state of plasticity, or to what is usually designated the normal consistency.

28.—Various methods have been proposed for making this determination, none of which has been found entirely satisfactory. The Committee recommends the following:

29.—*Method. Vicat Needle Apparatus.*—This consists of a frame (K), Fig. 2, bearing a movable rod (L), with the cap (A) at one end,

*The term "paste" is used in this report to designate a mixture of cement and water and the word "mortar" a mixture of cement, sand, and water.

and at the other the cylinder (B), 1 cm. (0.39 in.) in diameter, the cap, rod, and cylinder weighing 300 g. (10.58 oz.). The rod, which can be held in any desired position by a screw (F), carries an indicator, which moves over a scale (graduated to centimeters) attached to the frame (K). The paste is held by a conical, hard-rubber ring (I), 7 cm. (2.76 in.) in diameter at the base, 4 cm. (1.57 in.) high, resting on a glass plate (J), about 10 cm. (3.94 in.) square.

30.—In making the determination, the same quantity of cement as will be subsequently used for each batch in making the briquettes, but not less than 500 g., is kneaded into a paste, as described in Paragraph 56, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained 6 in. apart: the ball is then pressed into the rubber ring, through the larger opening, smoothed off, and placed (on its large end) on a glass plate and the smaller end smoothed off with a trowel; the paste, confined in the ring, resting on the plate, is placed under the rod bearing the cylinder, which is brought in contact with the surface and quickly released.

31.—The paste is of normal consistency when the cylinder penetrates to a point in the mass 10 mm. (0.39 in.) below the top of the ring. Great care must be taken to fill the ring exactly to the top.

32.—The trial pastes are made with varying percentages of water until the correct consistency is obtained.

33.—The Committee has recommended, as normal, a paste, the consistency of which is rather wet, because it believes that variations in the amount of compression to which the briquette is subjected in moulding are likely to be less with such a paste.

34.—Having determined in this manner the proper percentage of water required to produce a paste of normal consistency, the proper percentage required for the mortars is obtained from an empirical formula.

35.—The Committee hopes to devise such a formula. The subject proves to be a very difficult one, and, although the Committee has given it much study, it is not yet prepared to make a definite recommendation.

Time of Setting.

36.—*Significance.*—The object of this test is to determine the time which elapses from the moment water is added until the paste ceases to be fluid and plastic (called the "initial set"), and also the time required for it to acquire a certain degree of hardness (called the "final" or "hard set"). The former of these is the more important, since, with the commencement of setting, the process of crystallization or hardening is said to begin. As a disturbance of this process may produce a loss of strength, it is desirable to complete the operation of

mixing and moulding or incorporating the mortar into the work before the cement begins to set.

37.—It is usual to measure arbitrarily the beginning and end of the setting by the penetration of weighted wires of given diameters.

38.—*Method.*—For this purpose the Vicat Needle, which has already been described in Paragraph 29, should be used.

39.—In making the test, a paste of normal consistency is moulded and placed under the rod (*L*), Fig. 2, as described in Paragraph 30; this rod, bearing the cap (*D*) at one end and the needle (*H*), 1 mm. (0.039 in.) in diameter, at the other, weighing 300 g. (10.58 oz.). The needle is then carefully brought in contact with the surface of the paste and quickly released.

40.—The setting is said to have commenced when the needle ceases to pass a point 5 mm. (0.20 in.) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

41.—The test pieces should be stored in moist air during the test; this is accomplished by placing them on a rack over water contained in a pan and covered with a damp cloth, the cloth to be kept away from them by means of a wire screen; or they may be stored in a moist box or closet.

42.—Care should be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point reduces the area and tends to increase the penetration.

43.—The determination of the time of setting is only approximate, being materially affected by the temperature of the mixing water, the temperature and humidity of the air during the test, the percentage of water used, and the amount of moulding the paste receives.

Standard Sand.

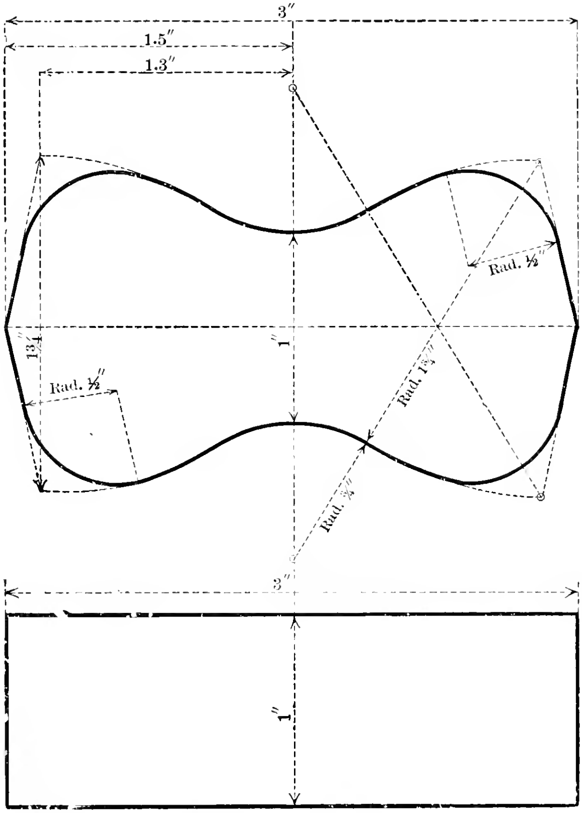
44.—The Committee recognizes the grave objections to the standard quartz now generally used, especially on account of its high percentage of voids, the difficulty of compacting in the moulds, and its lack of uniformity; it has spent much time in investigating the various natural sands which appeared to be available and suitable for use.

45.—For the present, the Committee recommends the natural sand from Ottawa, Ill., screened to pass a sieve having 20 meshes per linear inch and retained on a sieve having 30 meshes per linear inch; the wires to have diameters of 0.0165 and 0.0112 in., respectively, *i. e.*, half the width of the opening in each case. Sand having passed the No. 20 sieve shall be considered standard when not more than 1% passes a No. 30 sieve after one minute's continuous sifting of a 500-g. sample.*

* The Sandusky Portland Cement Company, of Sandusky, Ohio, has agreed to undertake the preparation of this sand, and to furnish it at a price only sufficient to cover the actual cost of preparation.

FORM OF BRIQUETTE.

46.—While the form of the briquette recommended by a former Committee of the Society is not wholly satisfactory, this Committee is not prepared to suggest any change, other than rounding off the corners by curves of ½-in. radius, Fig. 3.



DETAILS FOR BRIQUETTE.

FIG. 3.

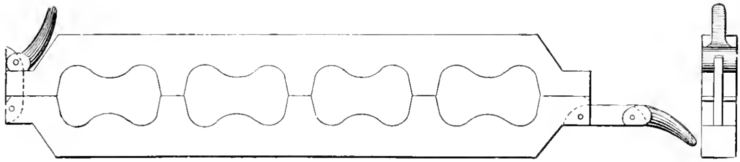
MOULDS.

47.—The moulds should be made of brass, bronze, or some equally non-corrodible material, having sufficient metal in the sides to prevent spreading during moulding.

48.—Gang moulds, which permit moulding a number of briquettes at one time, are preferred by many to single moulds; since the greater

quantity of mortar that can be mixed tends to produce greater uniformity in the results. The type shown in Fig. 4 is recommended.

49.—The moulds should be wiped with an oily cloth before using.



DETAILS FOR GANG MOULD

Fig. 4.

Mixing.

50.—All proportions should be stated by weight; the quantity of water to be used should be stated as a percentage of the dry material.

51.—The metric system is recommended because of the convenient relation of the gramme and the cubic centimeter.

52.—The temperature of the room and the mixing water should be as near 21° cent. (70° Fahr.) as it is practicable to maintain it.

53.—The sand and cement should be thoroughly mixed dry. The mixing should be done on some non-absorbing surface, preferably plate glass. If the mixing must be done on an absorbing surface it should be thoroughly dampened prior to use.

54.—The quantity of material to be mixed at one time depends on the number of test pieces to be made; about 1 000 g. (35.28 oz.) makes a convenient quantity to mix, especially by hand methods.

55.—The Committee, after investigation of the various mechanical mixing machines, has decided not to recommend any machine that has thus far been devised, for the following reasons:

(1) The tendency of most cement is to "ball up" in the machine, thereby preventing the working of it into a homogeneous paste; (2) there are no means of ascertaining when the mixing is complete without stopping the machine, and (3) the difficulty of keeping the machine clean.

56.—*Method.*—The material is weighed and placed on the mixing table, and a crater formed in the center, into which the proper percentage of clean water is poured; the material on the outer edge is turned into the crater by the aid of a trowel. As soon as the water has been absorbed, which should not require more than one minute, the operation is completed by vigorously kneading with the hands for an additional one minute, the process being similar to that used in kneading dough. A sand-glass affords a convenient guide for the time of

kneading. During the operation of mixing, the hands should be protected by gloves, preferably of rubber.

Moulding.

57.—Having worked the paste or mortar to the proper consistency, it is at once placed in the moulds by hand.

58.—The Committee has been unable to secure satisfactory results with the present moulding machines; the operation of machine moulding is very slow, and the present types permit of moulding but one briquette at a time, and are not practicable with the pastes or mortars herein recommended.

59.—*Method.*—The moulds should be filled immediately after the mixing is completed, the material pressed in firmly with the fingers and smoothed off with a trowel without mechanical ramming; the material should be heaped up on the upper surface of the mould, and, in smoothing off, the trowel should be drawn over the mould in such a manner as to exert a moderate pressure on the excess material. The mould should be turned over and the operation repeated.

60.—A check upon the uniformity of the mixing and moulding is afforded by weighing the briquettes just prior to immersion, or upon removal from the moist closet. Briquettes which vary in weight more than 3% from the average should not be tested.

Storage of the Test Pieces.

61.—During the first 24 hours after moulding, the test pieces should be kept in moist air to prevent them from drying out.

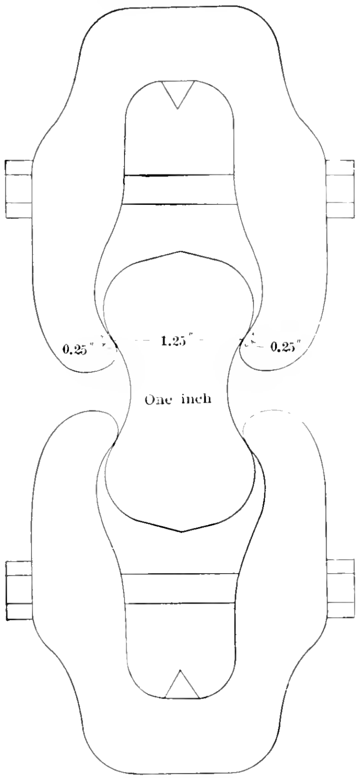
62.—A moist closet or chamber is so easily devised that the use of the damp cloth should be abandoned if possible. Covering the test pieces with a damp cloth is objectionable, as commonly used, because the cloth may dry out unequally, and, in consequence, the test pieces are not all maintained under the same condition. Where a moist closet is not available, a cloth may be used and kept uniformly wet by immersing the ends in water. It should be kept from direct contact with the test pieces by means of a wire screen or some similar arrangement.

63.—A moist closet consists of a soapstone or slate box, or a metal-lined wooden box—the metal lining being covered with felt and this felt kept wet. The bottom of the box is so constructed as to hold water, and the sides are provided with cleats for holding glass shelves on which to place the briquettes. Care should be taken to keep the air in the closet uniformly moist.

64.—After 24 hours in moist air, the test pieces for longer periods of time should be immersed in water maintained as near 21° cent. (70° Fahr.) as practicable; they may be stored in tanks or pans, which should be of non-corrodible material.

Tensile Strength.

65.—The tests may be made on any standard machine. A solid metal clip, as shown in Fig. 5, is recommended. This clip is to be used without cushioning at the points of contact with the test specimen. The bearing at each point of contact should be $\frac{1}{4}$ in. wide, and the distance between the center of contact on the same clip should be $1\frac{1}{4}$ in.



FORM OF CLIP.
Fig. 5.

66.—Test pieces should be broken as soon as they are removed from the water. Care should be observed in centering the briquettes in the testing machine, as cross-strains, produced by improper centering, tend to lower the breaking strength. The load should not be applied too suddenly, as it may produce vibration, the shock from which often breaks the briquette before the ultimate strength is reached. Care must be taken that the clips and the sides of the briquette be clean and free from grains of sand or dirt, which would prevent a good bearing. The load should be applied at the rate of 600 lb. per min. The average of the briquettes of each sample tested should be taken as the test, excluding any results which are manifestly faulty.

Constancy of Volume.

67.—*Significance.*—The object is to develop those qualities which tend to destroy the strength and durability of a cement. As it is highly essential to determine such qualities at once, tests of this character are for the most part made in a very short time, and are known, therefore, as accelerated tests. Failure is revealed by cracking, checking, swelling, or disintegration, or all of these phenomena. A cement which remains perfectly sound is said to be of constant volume.

68.—*Methods.*—Tests for constancy of volume are divided into two classes: (1) normal tests, or those made in either air or water maintained at about 21° cent. (70° Fahr.), and (2) accelerated tests,

APPARATUS FOR MAKING ACCELERATED TEST FOR SOUNDNESS OF CEMENT.

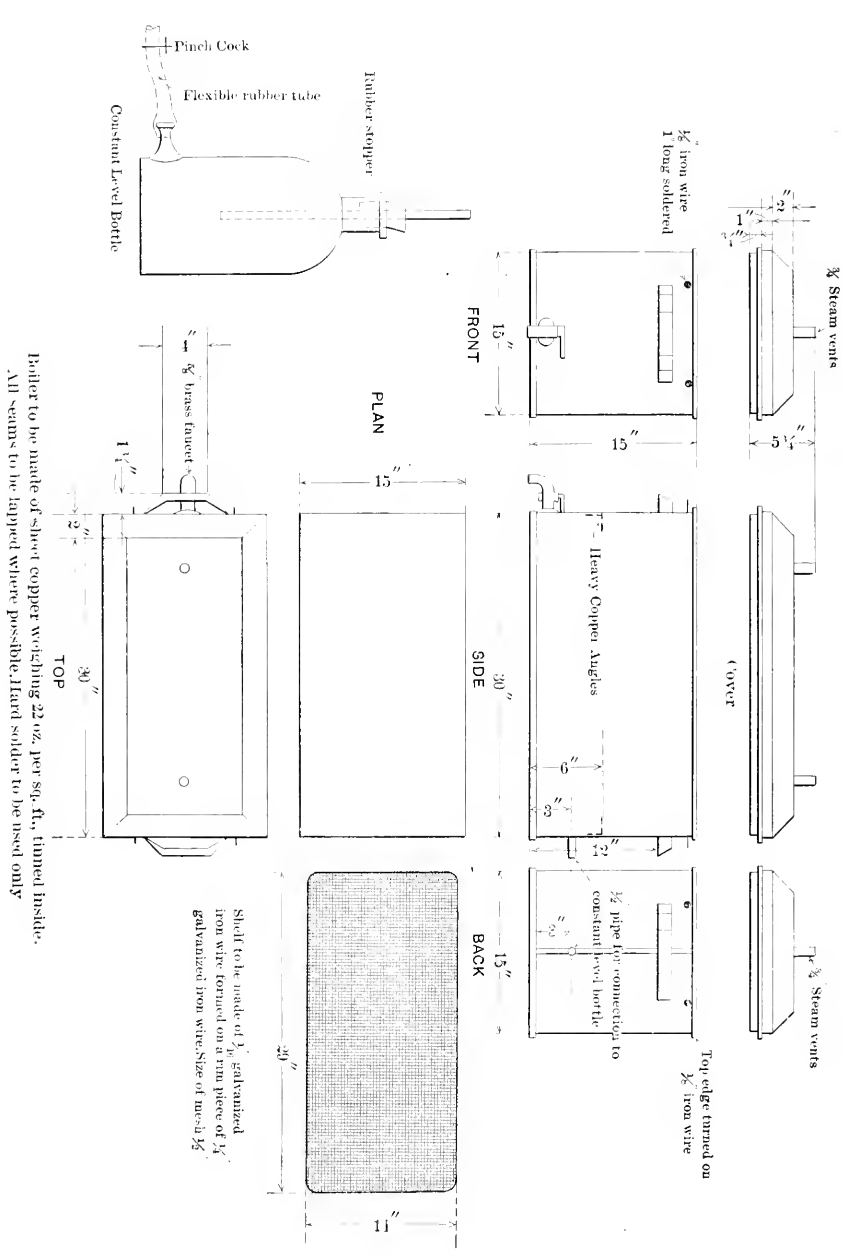


FIG. 6.

or those made in air, steam, or water at a temperature of 45° cent. (113° Fahr.) and upward. The test pieces should be allowed to remain 24 hours in moist air before immersion in water or steam, or preservation in air.

69.—For these tests, pats, about 7½ cm. (2.95 in.) in diameter, 1¼ cm. (0.49 in.) thick at the center, and tapering to a thin edge, should be made, upon a clean glass plate [about 10 cm. (3.94 in.) square], from cement paste of normal consistency.

70.—*Normal Test.*—A pat is immersed in water maintained as near 21° cent. (70° Fahr.) as possible for 28 days, and observed at intervals. A similar pat, after 24 hours in moist air, is maintained in air at ordinary temperature and observed at intervals.

71.—*Accelerated Test.*—A pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel, for 5 hours. The apparatus recommended for making these determinations is shown in Fig. 6.

72.—To pass these tests satisfactorily, the pats should remain firm and hard, and show no signs of cracking, distortion or disintegration.

73.—Should the pat leave the plate, distortion may be detected best with a straight-edge applied to the surface which was in contact with the plate.

74.—In the present state of our knowledge it cannot be said that cement should necessarily be condemned simply for failure to pass the accelerated tests; nor can a cement be considered entirely satisfactory simply because it has passed these tests.

The above is a revision of the Progress Report of your Committee presented at the Annual Meeting, January 21st, 1903, and embodying all the changes which have since been suggested from time to time.

Matters under investigation, therein referred to, have been considered, as follows:

(1) The determination of the character, size, and shape of grains necessary for uniformity in Standard Sand;

(2) The form of briquette and clips to be used in tension tests, and the form of test pieces in compression tests;

(3) The proper method of making compression tests;

(4) The determination of the proper percentage of water for use in sand mixtures;

(5) The best method for the determination of soundness or constancy of volume.

The progress made has not been as great as was expected, and your Committee finds itself unable to present a report on the above named subjects at this time. In the matter of Standard Sand, it feels confident that it will be able to present a report at the next Annual Meeting, and asks to be continued.

Submitted on behalf of the Committee,

George S. Webster,
Chairman.
Richard L. Humphrey,
Secretary.

December 28th, 1908.

Committee.
George S. Webster.
Richard L. Humphrey.
George F. Swain,
Alfred Noble,
Louis C. Sabin.
S. B. Newberry.
Clifford Richardson.
W. B. W. Howe.
F. H. Lewis.

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Presented to the Annual Meeting, January 20th, 1909.

"This Report has not yet been Placed Before or Accepted By the Society. Its Discussion has been made an Order of Business for the Next Annual Convention, July 6th-9th, 1909, and Written Communications concerning it Received Before that Date, will be Published in 'Proceedings'. All Rights to Publication are Reserved."

MAJORITY REPORT.*

I. Introduction.

Special Committees were appointed by the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers, for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete and to recommend necessary factors and formulas required in the design of structures in which these materials are used.

At the Annual Convention of the American Society of Civil Engineers held at Asheville, N. C., June 11th, 1903, the following resolution was adopted:

It is the sense of this meeting that a Special Committee be appointed to take up the question of concrete and steel-concrete, and that such committee co-operate with the American Society for Testing Materials, and the American Railway Engineering and Maintenance of Way Association.

Following the adoption of this resolution, a Special Committee on concrete and steel-concrete was appointed by the Board of Direction on May 31st, 1904. At the Annual Meeting, held January 18th, 1905, the title of this special committee was, at the request of the Committee, changed to "Special Committee on Concrete and Reinforced Concrete."

At the annual meeting of the American Society for Testing Materials held July 1st, 1903, at the Delaware Water Gap, the following resolution was unanimously adopted:

That the Executive Committee be requested to consider the desirability of appointing a committee on "Reinforced Concrete," with a view of co-operating with the committees of other societies in the study of the subject.

At the meeting of the Executive Committee of that Society, held December 5th, 1903, a special committee on "Reinforced Concrete" was appointed.

* For Minority Report, see page 119.

The American Railway Engineering and Maintenance of Way Association appointed a Committee on Masonry on July 20th, 1899, with instructions, as a part of its duties, to prepare specifications for concrete masonry. A preliminary set of specifications for Portland cement concrete was reported to and adopted by the Association on March 19th, 1903. At the meeting held in Chicago on March 17th, 1904, the Committee on Masonry was authorized to co-operate with the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, and following this action a special subcommittee was appointed.

At a meeting of the several special committees representing the above-mentioned societies, held at Atlantic City, N. J., June 17th, 1904, arrangements were completed for collaborating the work of these several committees through the formation of the Joint Committee on Concrete and Reinforced Concrete. Mr. C. C. Schneider was elected temporary chairman and Professor A. N. Talbot was elected temporary secretary. The proposed plan of action of the special committee of the American Society of Civil Engineers was outlined, involving the appointment of subcommittees on Plan and Scope, on Tests, and on Ways and Means.

Mr. Schneider and Mr. Schaub, as Chairman and Secretary of the Special Committee of the American Society of Civil Engineers, were elected permanent Chairman and Secretary of the Joint Committee on Concrete and Reinforced Concrete. Mr. Emil Swensson was elected Vice-Chairman, and, on the resignation of Mr. Schaub, Mr. Richard L. Humphrey was elected Secretary.

The following is the organization of the Joint Committee:

OFFICERS.

Chairman—C. C. SCHNEIDER.
Vice-Chairman—EMIL SWENSSON.
Secretary—RICHARD L. HUMPHREY.

MEMBERS.

American Society of Civil Engineers (Special Committee on Concrete and Reinforced Concrete):

Greiner, J. E., Consulting Engineer, Baltimore and Ohio Railroad, Baltimore, Md.

Hatt, W. K., Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Hoff, Olaf, Vice-President, Butler Brothers, Hoff and Company, New York, N. Y.

Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.

Lesley, R. W., President, American Cement Company, Philadelphia, Pa.

Schaub, J. W., Consulting Engineer, Chicago, Ill.

Schneider, C. C., Consulting Engineer, Philadelphia, Pa.
Swensson, Emil, Consulting Engineer, Pittsburg, Pa.
Talbot, A. N., Professor of Municipal and Sanitary Engineering, Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.
Worcester, J. R., Consulting Engineer, Boston, Mass.

American Society for Testing Materials (Committee on Reinforced Concrete):

Fuller, William B., Consulting Engineer, New York, N. Y.
Heidenreich, E. Lee, Consulting Engineer, New York, N. Y.
Humphrey, Richard L., Consulting Engineer, Engineer in Charge, Structural Materials Testing Laboratories, U. S. Geological Survey, Philadelphia, Pa.
Johnson, Albert L., Consulting Engineer, St. Louis, Mo.
Lanza, Gaetano, Professor of Theoretical and Applied Mechanics, Massachusetts Institute of Technology, Boston, Mass.
Lesley, R. W., President, American Cement Company, Philadelphia, Pa.
Marburg, Edgar, Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.
Mills, Charles M., Principal Assistant Engineer, Philadelphia Rapid Transit Company, Philadelphia, Pa.
Moisseiff, Leon S., Assistant Engineer, Department of Bridges, New York, N. Y.
Quimby, Henry H., Assistant Engineer of Bridges, Bureau of Surveys, Philadelphia, Pa.
Taylor, W. P., Engineer in Charge of Testing Laboratory, Philadelphia, Pa.
Thompson, Sanford E., Consulting Engineer, Newton Highlands, Mass.
Turneaure, F. E., Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.
Wagner, Samuel Tobias, Assistant Engineer, Philadelphia and Reading Railroad, Philadelphia, Pa.
Webster, George S., Chief Engineer, Bureau of Surveys, Philadelphia, Pa.

American Railway Engineering and Maintenance of Way Association (Subcommittee on Reinforced Concrete):

Boynton, C. W., Inspecting Engineer, Universal Portland Cement Company, Chicago, Ill.
Cunningham, A. O., Chief Engineer, Wabash Railroad, St. Louis, Mo.
Moore, C. H., Engineer of Grade Crossings, Erie Railroad, New York, N. Y.
Scribner, Gilbert H., Jr., Contracting Engineer, Chicago, Ill.
Swain, George F., Professor of Civil Engineering, Massachusetts Institute of Technology, Boston, Mass.

Association of American Portland Cement Manufacturers (Committee on Concrete and Steel Concrete):

Fraser, Norman D., President, Chicago Portland Cement Company, Chicago, Ill.

Griffiths, R. E., Vice-President, American Cement Company, Philadelphia, Pa.

Hagar, Edward M., President, Universal Portland Cement Company, Chicago, Ill.

Newberry, Spencer B., Manager, Sandusky Portland Cement Company, Sandusky, Ohio.

The report, herein presented, embodies the present judgment of the Committee concerning the proper use of Concrete and Reinforced Concrete.

II. Adaptability of Concrete and Reinforced Concrete.

The adaptability of concrete and reinforced concrete for engineering structures, or parts thereof, is now so well established that it may be considered one of the recognized materials of construction. It has proved to be a satisfactory material, when properly used, for those purposes for which its qualities make it particularly suitable.

1. Proper Use.

Concrete is a material of very low tensile strength and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly upon its durability, its fire-resisting qualities, its strength in compression and its relatively low cost. Its strength increases generally with age.

Plain concrete or massive concrete is well adapted for structural forms in which the principal stresses are compressive. These include foundations, dams, retaining and other walls, piers, abutments, short columns and, in many cases, arches. In the design of massive concrete, the tensile strength of the material must generally be neglected.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and metal is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. In structures resisting lateral forces it possesses advantages over plain concrete in that it may be so designed as to utilize more fully the strength rather than the weight of the material.

2. Improper Use.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: defective design, poor material, and faulty execution.

The defects in a design may be many and various. The computations and assumptions on which they are based may be faulty and contrary to the established principles of statics and mechanics; the unit stresses used may be excessive, or the details of the design defective.

The design of reinforced concrete structures should receive at least the same careful consideration as those of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts, so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections, accompanied by computations to prove their strength.

The use of high unit stresses, approaching the danger line, is a defect in the design of reinforced concrete structures.

Articulated concrete structures, designed in imitation of steel trusses, may be mentioned as illustrating a questionable use of reinforced concrete.

Poor material is sometimes used for the concrete, as well as for the reinforcement. The use of inferior concrete is generally due to a lack of experience of the contractor and his superintendents, or to the absence of proper supervision.

An unsuitable quality of steel for reinforcement is sometimes prescribed in specifications, for the purpose of reducing the cost. For steel structures, a high grade of material is specified, while the steel used for reinforcing concrete is sometimes made of unsuitable, brittle material.

Faulty execution and careless workmanship may generally be attributed to unintelligent or insufficient supervision.

While other engineering structures, upon the safety of which human lives depend, are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures frequently are designed by contractors or by engineers commercially interested, and the contract let for a lump sum.

The construction of buildings in large cities is regulated by ordinances or building laws, and the work is inspected by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Means for more adequate supervision and inspection should, therefore, be provided.

3. Responsibility and Supervision.

The execution of the work should not be separated from the design, since intelligent supervision and successful execution can be expected only when both functions are combined. The engineer who prepares the design and specifications should therefore have the supervision of the execution of the work.

The Committee recommends the following rules for structures of reinforced concrete, for the purpose of fixing the responsibility and providing for adequate supervision during construction:

a. Before work is commenced, complete plans shall be prepared, accompanied by specifications, static computations and descriptions showing the general arrangement and all details. The static computations shall give the loads assumed separately, such as dead and live loads, wind and impact, if any, and the resulting stresses.

b. The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

c. The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

d. The drawings and specifications shall be signed by the engineer and the contractor.

e. The approval of plans and specifications by other authorities shall not relieve the engineer nor the contractor of responsibility.

f. Inspection during construction shall be made by competent inspectors employed by, and under the supervision of, the engineer, and shall cover the following:

1. The materials.
2. The correct construction and erection of the forms and the supports.
3. The sizes, shapes and arrangement of the reinforcement.
4. The proportioning, mixing and placing of the concrete.
5. The strength of the concrete by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Prevention of injury to any part of the structure by and after the removal of the forms.
8. Comparison of dimensions of all parts of the finished structure with the plans.

g. Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. Loading shall be carried to such a point that twice the calculated working stresses in critical parts are reached, and such loads shall cause no permanent deformations. Load tests shall not be made until after 60 days of hardening.

4. Destructive Agencies.

a. Corrosion of Metal Reinforcement.—Tests and experience have proved that steel embedded in good concrete will not corrode, no matter whether located above or below fresh or sea water level. If the

concrete is porous, so as to be readily permeable to water, as, where the concrete is laid with a very dry consistency, the metal may be corroded in the presence of moisture.

b. Electrolysis.—There is little accurate information available as to the effect of electrolysis on concrete. The few experiments that are available seem to indicate that concrete may be damaged through the leakage of small electrical currents through the mass, particularly where steel is embedded in the concrete. These experiments are not conclusive, however, and the large numbers of reinforced concrete structures subject to the action of electrolysis, in which the metal and concrete are in perfect condition, would seem to indicate that the destructive action reported was due to abnormal conditions which do not often occur in practice.

c. Salt Water.—The data available concerning the effect of sea water on concrete or reinforced concrete are inconclusive and limited in amount. There have been no authentic cases reported where the disintegration has proved to be due entirely to sea water. The decomposition that has been reported manifests itself in a number of ways; in some cases the mortar softens and crumbles; in others, a crust forms which in time comes off. It has been found, however, that where concrete is proportioned in such a way as to secure a maximum density and is mixed thoroughly it makes an impervious concrete, upon which sea water has apparently little effect. Sea-walls have been standing for considerable lengths of time without apparent injury. In many of our harbors where the water has been rendered brackish through the rivers discharging into them, the action that has been reported has been at the water line and was probably due in part to freezing.

d. Acids.—Concrete of first-class quality, thoroughly hardened, is affected appreciably only by strong acids which seriously injure other materials. A substance like manure, because of the acid in its composition, is injurious to green concrete, but after the concrete has thoroughly hardened it satisfactorily resists such action.

e. Oils.—When concrete is properly made and the surface carefully finished and hardened it resists the action of such oils as petroleum and ordinary engine oils. Certain oils which contain fatty acids appear to produce injurious effects.

f. Alkalies.—The action of alkalies on concrete is problematical. In the reclamation of arid land, where the soil is heavily charged with alkaline salts, it has been found that concrete, stone, brick, iron and other materials are injured under certain conditions. It would seem that at the level of the ground-water such structures are disintegrated, possibly due in part to the effect of formation of crystals resulting from the alternate wetting and drying of the surface of the concrete at this ground-water line. Such destructive action can be prevented by the use of an insulating coating.

III. Materials.

A knowledge of the properties of the materials entering into concrete and reinforced concrete is the first essential. The importance of the quality of the materials used cannot be over-estimated, and, not only the cement, but also the aggregates, should be subject to such definite requirements and tests as will insure a concrete of the required quality.

1. Cement.

There are available, for construction purposes, Portland, Natural, and Puzzolan or Slag cements. Only Portland cement is suitable for reinforced concrete.

a. Portland Cement is the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials. It has a definite chemical composition varying within comparatively narrow limits.

Portland cement should be used in reinforced concrete construction and any construction that will be subject to shocks or vibrations or stresses other than direct compression.

b. Natural Cement is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas. While the limestone must have a certain composition, this composition may vary in much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly nor is it as uniform in composition as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor, a comparison may be made between the use of Natural cement and a leaner mixture of Portland cement that will develop the same strength.

c. Puzzolan or Slag Cement is the finely pulverized product resulting from grinding a mechanical mixture of granulated basic blast-furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or Natural cement, is not extensively used, and never in important work; it should be used only for foundation work underground where it is not exposed to air or running water.

d. Specifications. The cement should meet the requirements of the Standard Specifications for Cement (see Appendix, p. 108). A number of societies have been working on methods for testing and specifications for cement. The best practice seems to be represented in the standard methods of testing and specifications for cement which are the result of the joint labors of Special Committees of the American Society of Civil Engineers, American Society for Testing Materials,

American Railway Engineering and Maintenance of Way Association, American Institute of Architects, and others.

2. Aggregates.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

a. Fine Aggregate consists of sand, crushed stone, or gravel screenings, passing when dry a screen having ¼-in. diameter holes. It should be preferably of silicious material, clean, coarse, free from vegetable loam or other deleterious matter.

A gradation of the grain from fine to coarse is generally advantageous.

Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

b. Coarse Aggregate consists of inert material, such as crushed stone, or gravel, which is retained on a screen having ¼-in. diameter holes. The particles should be clean, hard, durable, and free from all deleterious material. Aggregates containing soft, flat or elongated particles should be excluded from important structures. A gradation of sizes of the particles is generally advantageous.

The maximum size of the coarse aggregate shall be such that it will not separate from the mortar in laying and will not prevent the concrete from fully surrounding the reinforcement and filling all parts of the forms. Where concrete is used in mass, the size of the coarse aggregate may be such as to pass a 3-in. ring. For reinforced members a size to pass a 1-in. ring, or a smaller size, may be used.

Cinder concrete is not suitable for reinforced concrete structures, and may be safely used only in mass for very light loads or for fireproofing.

Where cinder concrete is permissible, the cinders used as the coarse aggregate should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes.

3. Water.

The water used in mixing concrete should be free from oil, acid, strong alkalies, or vegetable matter.

* A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

4. Metal Reinforcement.

The Committee recommends, as a suitable material for reinforcement, steel filling the requirements of the specifications adopted by the American Railway Engineering and Maintenance of Way Association (Appendix, p. 112).

For the reinforcement of slabs, small beams, or minor details, or for the prevention of shrinkage cracks where wire or small rods are suitable, material conforming to the requirements of either Specification A or B given in the Appendix (pages 113 and 114) may be used.

The reinforcement should be free from rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

IV. Preparation and Placing of Mortar and Concrete.

1. Proportions.

The materials to be used in concrete should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

a. Unit of Measure.—The unit of measure should be the barrel, which should be taken as containing 3.8 cu. ft. Four bags containing 94 lb. of cement each should be considered the equivalent of one barrel. Fine and coarse aggregate should be measured separately as loosely thrown into the measuring receptacle.

b. Relation of Fine and Coarse Aggregates.—The fine and coarse aggregates should be used in such relative proportions as will insure maximum density. In unimportant work it is sufficient to do this by individual judgment, using correspondingly higher proportions of cement; for important work these proportions should be carefully determined by density experiments, and the sizing of the fine and coarse aggregates should be uniformly maintained, or the proportions changed to meet the varying sizes.

c. Relation of Cement and Aggregates.—For reinforced concrete construction, a density proportion based on 1: 6 should generally be used, *i. e.,* one part of cement to a total of six parts of fine and coarse aggregates measured separately.

In columns, richer mixtures are often required, while for massive masonry or rubble concrete a leaner mixture, of 1: 9 or even 1: 12, may be used. These proportions should be determined by the strength or wearing qualities required in the construction at the critical period of its use. Experienced judgment based on individual observation and tests of similar conditions in similar localities is the best guide as to the proper proportions for any particular case.

2. Mixing.

The ingredients of concrete should be thoroughly mixed to the desired consistency, and the mixing should continue until the cement

is uniformly distributed and the mass is uniform in color and homogeneous, since the maximum density and therefore the greatest strength of a given mixture depends largely on thorough and complete mixing.

a. Measuring Ingredients.—Methods of measurement of the proportions of the various ingredients, including the water, should be used, which will secure separate uniform measurements at all times.

b. Machine Mixing.—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, since a more thorough and uniform consistency can be thus obtained.

c. Hand Mixing.—When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn the materials until they are homogeneous in appearance and color.

d. Consistency.—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement, and, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

e. Retempering.—Retempering mortar or concrete, *i. e.*, remixing with water after it has partially set, should not be permitted.

3. Placing of Concrete.

a. Methods.—Concrete, after the addition of water to the mix, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances should concrete be used that has partially set before final placing. A slow-setting cement should be used when a long time is likely to occur between mixing and final placing.

The concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place by gravity and the surplus water has been forced to the surface.

In depositing the concrete under water, special care should be exercised to prevent the cement from being floated away, and to prevent the formation of laitance which hardens very slowly and forms a poor surface on which to deposit fresh concrete. Laitance is formed in both still and running water, and should be removed before placing fresh concrete.

Before placing the concrete, care should be taken to see that the forms are substantial and thoroughly wetted and the space to be occupied by the concrete is free from débris. When the placing of the concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The faces of concrete exposed to premature drying should be kept wet for a period of at least seven days.

b. Freezing Weather.—Concrete for reinforced structures should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

c. Rubble Concrete.—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced through the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

V. Forms.

Forms should be substantial and unyielding, so that the concrete shall conform to the designed dimensions and contours, and should be tight to prevent the leakage of mortar.

The time for removal of forms is one of the most important steps in the erection of a structure of concrete or reinforced concrete. Care should be taken to inspect the concrete and ascertain its hardness before removing the forms.

So many conditions affect the hardening of concrete that the proper time for the removal of the forms should be decided by some competent and responsible person, especially where the atmospheric conditions are unfavorable.

VI. Details of Construction.

1. Joints.

a. Reinforcement.—Wherever in tension reinforcement it is necessary to splice the reinforcing bars, the length of lap shall be determined on the basis of the safe bond stress and the stress in the bar at the point of splice; or a connection shall be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress should be avoided. In columns, large bars should be properly butted and spliced; small bars may be treated as indicated for tension reinforcement, or their stress may be taken off by being embedded in large masses of concrete. At foundations, bearing plates should be provided for large bars or structural forms.

b. Concrete.—For concrete construction it is desirable to cast the entire structure at one operation, but as this is not always possible,

especially in large structures, it is necessary to stop the work at some convenient point. This point should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders; that the joints in girders be at a point midway between supports, but should a beam intersect a girder at this point, the joint should be offset a distance equal to twice the width of the beam; that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis of the column, and in girders, beams and floor slabs perpendicular to the plane of their surfaces.

2. Shrinkage.

Girders should never be constructed over freshly formed columns without permitting a period of at least two hours to elapse, thus providing for settlement or shrinkage in the columns. Before resuming work, the top of the column should be thoroughly cleansed of foreign matter and laitance. If the concrete in the column has become hard, the top should also be drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate before placing additional concrete.

3. Temperature Changes.

Concrete is sensitive to temperature changes, and it is necessary to take this fact into account in designing and erecting concrete structures. In some positions the concrete is subjected to a much greater fluctuation in temperature than in others, and in such cases joints are necessary. The frequency of these joints will depend, first, upon the range of temperature to which concrete will be subjected; second, upon the quantity and position of the reinforcement. These points should be determined and provided for in the design. In massive work, such as retaining walls, abutments, etc., built without reinforcement, joints should be provided, approximately, every 50 ft. throughout the length of the structure. To provide against the structures being thrown out of line by unequal settlement, each section of the wall may be tongued and grooved into the adjoining section. To provide against unsightly cracks, due to unequal settlement, a joint should be made at all sharp angles.

4. Fire-Proofing.

The actual fire tests of concrete and reinforced concrete have been limited, but experience, together with the results of tests so far made, indicate that concrete may be safely used for fire-proofing purposes. Concrete itself is incombustible and reasonably proof against fire when

composed of a silicious sand and a hard coarse aggregate such as igneous rock.

For a fire-proof covering, these same materials may be used, or clean hard-burned cinders may be substituted for the coarse aggregate.

The low rate of heat conductivity of concrete is one reason of its value for fire-proofing. The dehydration of the water of crystalization of concrete probably begins at about 500° Fahr., and is completed at about 900° Fahr., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire remains in position and affords protection to the concrete beneath it.

It is recommended that in monolithic concrete columns, the concrete to a depth of 1½ in. be considered as protective covering and not included in the effective section.

The thickness of the protective coating required depends upon the probable duration of a fire which is likely to occur in the structure, and should be based on the rate of heat conductivity. The question of the conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions it is recommended that the metal in girders and columns be protected by a minimum of 2 in. of concrete; that the metal in beams be protected by a minimum of 1½ in. of concrete, and that the metal in floor slabs be protected by a minimum of 1 in. of concrete.

It is recommended that the corners of columns, girders and beams be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one.

5. Water-Proofing.

Many expedients have been used to render concrete impervious to water under normal conditions, and also under pressure conditions that exist in reservoirs, dams, and conduits of various kinds. Experience shows, however, that where mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to a rather wet consistency, the resulting mortar or concrete is impervious under ordinary conditions. A concrete of dry consistency is more or less pervious to water, and compounds of various kinds have been mixed with the concrete, or applied as a wash to the surface for the purpose of making it water-tight. Many of these compounds are of but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls, and reservoirs, leakage cracks may be prevented by horizontal and vertical reinforcement,

properly proportioned and located, provided the concrete itself is impervious.

Such reinforcement distributes the stretch due to contraction or settlement so that the cracks are too minute to permit leakage, or are soon closed by infiltration of silt.

Asphaltic or coal-tar preparations, applied either as a mastic or as a coating on felt or cloth fabric, are used for water-proofing, and should be proof against injury by liquids or gases.

6. Surface Finish.

Concrete is a material of an individual type, and should not be used in imitation of other structural materials. One of the important problems connected with the use of concrete is the character of the finish of exposed surfaces. The finish of the surface should be determined before the concrete is placed, and the work conducted so as to make possible the finish desired. For many forms of construction the natural surface of the concrete is unobjectionable, but frequently the marks of the boards and the flat dead surface are displeasing, and make some special treatment desirable. A treatment of the surface which removes the film of mortar and brings the coarser particles of the concrete into relief is frequently used to remove the form markings, break the monotonous appearance of the surface, and make it more pleasing. Plastering of surfaces should be avoided, for the other methods of treatment are more reliable and usually much more satisfactory. Plastering, even if carefully applied, is likely to peel off under the action of frost or temperature changes.

VII. Design.

1. Massive Concrete.

In the design of massive concrete or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be so proportioned as to avoid tensile stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete will therefore often be suitable for massive concrete structures. Owing to its low extensibility, the contraction due to hardening and to temperature changes requires special consideration, and, except in the case of very massive walls such as dams, it is desirable to provide joints at intervals to localize the effect of such contrac-

tion. The spacing of such joints will depend upon the form and dimensions of the structure and its degree of exposure.

Massive concrete may well be used for piers and short columns, in which the ratio of length to least width is relatively small. Under ordinary conditions this ratio should not exceed six, but, where the central application of the load is assured, a somewhat higher value may safely be used.

Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable.

2. Reinforced Concrete.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and steel is particularly advantageous in the beam, where both compression and tension exist; it is also advantageous in the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design will therefore relate mainly to the analysis of beams and columns.

3. General Assumptions.

a. Loads.—The loads or forces to be resisted consist of:

1. The dead load, which includes the weight of the structure and fixed loads and forces.
2. The live load, or the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Any allowance for the dynamic effect is preferably taken into account by adding the desired amount to the live load or to the live-load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses so determined.

   In the case of high buildings the live load on columns may be reduced in accordance with the usual practice.

b. Lengths of Beams and Columns.—The span length for beams and slabs shall be taken as the distance from center to center of supports, but shall not be taken to exceed the clear span plus the depth of beam or slab. Brackets shall not be considered as reducing the clear span in the sense here intended.

The length of columns shall be taken as the maximum unsupported length.

c. Internal Stresses.—As a basis for calculations relating to the strength of structures, the following assumptions are recommended:

1. Calculations should be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression, within the usual limits of working stresses, is constant. The distribution of compressive stresses in beams is therefore rectilinear.
4. In calculating the moment of resistance of beams the tensile stresses in the concrete shall be neglected.
5. Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete may be taken at 15.
7. Initial stress in the reinforcement due to contraction or expansion in the concrete may be neglected.

It is appreciated that the assumptions herein given are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

For calculations relative to deflections, the tensile strength of the concrete should be taken into account. For such calculations, also, a value of 8 to 12 for the ratio of the moduli corresponds more nearly to the actual conditions and may well be used.

4. Tee-Beams.

In beam and slab construction, an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

Where adequate bond between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall be determined by the following rules:

a. It shall not exceed one-fourth of the span length of the beam;

b. Its overhanging width on either side of the web shall not exceed 4 times the thickness of the slab.

In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the support.

5. Floor Slabs.

Floor slabs should be designed and reinforced as continuous over the supports. If the length of the slab exceeds 1.5 times its width

the entire load should be carried by transverse reinforcement. Square slabs may well be reinforced in both directions.*

The loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beam, and may be assumed to vary in accordance with the ordinates of a triangle. The moments in the beams should be calculated accordingly.

6. Continuous Beams and Slabs.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

a. That for floor slabs the bending moments at center and at support be taken at $\frac{w\,l^2}{12}$ for both dead and live loads, where w represents the load per linear foot and l the span length.

b. That for beams the bending moment at center and at support for interior spans be taken at $\frac{w\,l^2}{12}$, and for end spans it be taken at $\frac{w\,l^2}{10}$ for center and adjoining support for both dead and live loads.

In the case of beams and slabs continuous for two spans only, or of spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress in accordance with

* The exact distribution of load on square and rectangular slabs, supported on four sides and reinforced in both directions cannot readily be determined. The following method of calculation is recognized to be faulty, but it is offered as a tentative method which will give results on the safe side. The distribution of load is first to be determined by the formula

$$r = \frac{l^4}{l^4 + b^4}$$

in which r = proportion of load carried by the transverse reinforcement, l = length and b = breadth of slab. For various ratios of l/b the values of r are as follows:

l/b	r
1	0.50
1.1	0.59
1.2	0.67
1.3	0.75
1.4	0.80
1.5	0.83

Using the values above specified, each set of reinforcement is to be calculated in the same manner as slabs having supports on two sides only, but the total amount of reinforcement thus determined may be reduced 25% by gradually increasing the rod spacing from the third point to the edge of the slab.

the provisions of Chap. VII, Section 3, *c*, paragraph 6. In the case of continuous beams, tensile and compressive reinforcement over supports must extend sufficiently beyond the support to develop the requisite bond strength.

7. Bond Strength and Spacing of Bars.

Adequate bond strength should be provided in accordance with the formula hereinafter given. Where a portion of the bars is bent up near the end of a beam, the bond stress in the remaining straight bars will be less than is represented by the theoretical formula.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than hooks consisting of turns through 180 degrees.

The lateral spacing of parallel bars should not be less than two and one-half diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than ½ in.

8. Shear and Diagonal Tension.

Calculations for web resistance shall be made on the basis of maximum shearing stress as determined by the formulas hereinafter given. When the maximum shearing stresses exceed the value allowed for the concrete alone, web reinforcement must be provided to aid in carrying the diagonal tension stresses. This web reinforcement may consist of bent bars, or inclined or vertical members attached to or looped about the horizontal reinforcement. Where inclined members are used, the connection to the horizontal reinforcement shall be such as to insure against slip.

Experiments bearing on the design of details of web reinforcement are not yet complete enough to allow more than general and tentative recommendations to be made. It is well established, however, that a very moderate amount of reinforcement, such as is furnished by a few bars bent up at small inclination, increases the strength of a beam against failure by diagonal tension to a considerable degree; and that a sufficient amount of web reinforcement can readily be provided to increase the shearing resistance to a value from three or more times that found when the bars are all horizontal and no web reinforcement is used. The following allowable values for the maximum shearing stress are therefore recommended, based on the working stresses of Section VIII, page 106.

a. For beams with horizontal bars, only 40 lb. per sq. in.

b. For beams in which a part of the horizontal reinforcement is used in the form of bent-up bars, arranged with due respect to the shearing stresses, a higher value may be allowed, but not to exceed 60 lb. per sq. in.

c. For beams thoroughly reinforced for shear, a value not exceeding 120 lb. per sq. in.

In the calculation of web reinforcement to provide the strength required under *c* above, the concrete may be counted upon as carrying one-third of the shear. The remainder is to be provided for by means of metal reinforcement consisting of bent rods or stirrups, but preferably both. The requisite amount of such reinforcement may be estimated on the assumption that the entire shear on a section, less the amount assumed to be carried by the concrete, is carried by the reinforcement in a length of beam equal to its depth.

The longitudinal spacing of stirrups or bent rods shall not exceed three-fourths the depth of the beam.

It is important that adequate bond strength be provided to develop fully the assumed strength of all shear reinforcement.

Inasmuch as small deformations in the horizontal steel tends to prevent the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by so arranging the horizontal reinforcement that the unit stresses at points of large shear shall be relatively low.

9. Columns.

It is recommended that the ratio of the unsupported length of a column to its least width be limited to 15.

The effective area of the column shall be taken as the area within the protective covering, as defined in Chap. VI, Section 4; or, in the case of hooped columns or columns reinforced with structural shapes, it shall be taken as the area within the hooping or structural shapes.

Columns may be reinforced by means of longitudinal rods, by bands or hoops, by bands or hoops together with longitudinal bars, or by structural forms which in themselves are sufficiently rigid to act as columns. The general effect of bands or hoops is to increase greatly the "toughness" of the column and its ultimate strength, but hooping has little effect upon its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of "toughening" are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the possible deformation before ultimate failure.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

a. Columns with longitudinal reinforcement only, the unit stress recommended for axial compression in Chap. VIII. Section 3.

b. Columns with reinforcement of bands or hoops, as hereinafter specified, stresses 20% higher than given for *a*.

c. Columns reinforced with not less than 1% and not more than 4% of longitudinal bars and with bands or hoops, stresses 45% higher than given for *a*.

d. Columns reinforced with structural steel column units which thoroughly encase the concrete core, stresses 45% higher than given for *a*.

In all cases, longitudinal steel is assumed to carry its proportion of stress in accordance with Section 3. The hoops or bands are not to be counted upon directly as adding to the strength of the column.

Bars composing longitudinal reinforcement shall be straight, and shall have sufficient lateral support to be securely held in place until the concrete has set.

Where bands or hoops are used, the total amount of such reinforcement shall not be less than 1% of the volume of the column disclosed. The clear spacing of such bands or hoops shall not be greater than one-fourth the diameter of the enclosed column. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered.

Bending stresses due to eccentric loads must be provided for by increasing the section until the maximum stress does not exceed the values above specified.

10. Reinforcing for Shrinkage and Temperature Stresses.

Where large areas of concrete are exposed to atmospheric conditions, the changes of form due to shrinkage (resulting from hardening) and to action of temperature are such that large cracks will occur in the mass, unless precautions are taken to so distribute the stresses as either to prevent the cracks altogether or to render them very small. The size of the cracks will be directly proportional to the diameter of the reinforcing bars and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement should be placed near the surface and well distributed, and a form of reinforcement used which will develop a high bond resistance.

VIII. Working Stresses.

1. General Assumptions.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress to be allowed on concrete, we should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class but composed of different materials may have approximately the same degree of safety.

The stresses for concrete are proposed for concrete composed of one part Portland cement and six parts of aggregates, capable of developing an average compressive strength of 2 000 lb. per sq. in. at 28 days, when tested in cylinders 8 in. in diameter and 16 in. long, under laboratory conditions of manufacture and storage, using the same consistency as is used in the field. In considering the factors recommended with relation to this strength, it is to be borne in mind that the strength at 28 days is by no means the ultimate which will be developed at a longer period, and therefore they do not correspond with the real factor of safety. On concretes, in which the material of the aggregate is inferior, all stresses should be proportionally reduced, and similar reduction should be made when leaner mixes are to be used. On the other hand, if, with the best quality of aggregates, the richness is increased, an increase may be made in all working stresses proportional to the increase in compressive strength at 28 days, but this increase shall not exceed 25 per cent.

2. Bearing.

When compression is applied to a surface of concrete larger than the loaded area, a stress of 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. on the above-described concrete, may be allowed. This pressure is probably unnecessarily low when the ratio of the stressed area to the whole area of the concrete is much below unity, but is recommended for general use rather than a variable unit based upon this ratio.

3. Axial Compression.

For concentric compression on a plain concrete column or pier, the length of which does not exceed 12 diameters, 22.50% of the compressive strength at 28 days, or 450 lb. per sq. in. on 2 000-lb. concrete, may be allowed.

For other forms of columns, the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

4. Compression in Extreme Fiber.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses, may be allowed to reach 32.5% of the compressive strength at 28 days, or 650 lb. per sq. in. for 2 000-lb. concrete. Adjacent to the support of continuous beams, stresses 15% higher may be used.

5. Shear and Diagonal Tension.

Where pure shearing stress occurs, that is, uncombined with compression normal to the shearing surface, and with all tension normal to the shearing plane provided for reinforcement, a shearing stress of 6% of the compressive strength at 28 days, or 120 lb. per sq. in. on 2 000-lb. concrete, may be allowed. Where the shear is combined with an equal compression, as on a section of a column at 45° with the axis, the stress may equal one-half the compressive stress allowed. For ratios of compressive stress to shear intermediate between 0 and 1, proportionate shearing stresses shall be used.

In calculations on beams in which diagonal tension is considered to be taken by the concrete, the vertical shearing stresses should not exceed 2% of the compressive strength at 28 days, or 40 lb. per sq. in. for 2 000-lb. concrete.

6. Bond.

The bonding stress between concrete and plain reinforcing bars may be assumed at 4% of the compressive strength at 28 days, or 80 lb. per sq. in. for 2 000-lb. concrete; in the case of drawn wire, 2% or 40 lb. on 2 000-lb. concrete.

7. Reinforcement.

The tensile stress in steel should not exceed 16 000 lb. per sq. in. The compressive stress in reinforcing steel should not exceed 16 000 lb. per sq. in., or 15 times the working compressive stress in the concrete.

In structural steel members, the working stresses adopted by the American Railway Engineering and Maintenance of Way Association are recommended.

8. Modulus of Elasticity.

The value of the modulus of elasticity of concrete has a wide range, depending upon the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in all computations it be assumed as one-fifteenth that of steel, as, while not rigorously accurate, this assumption will give safe results.

RICHARD L. HUMPHREY,
Secretary.

JANUARY, 1909.

W. K. HATT.
R. W. LESLEY.
A. N. TALBOT.
J. W. SCHAUB.
J. R. WORCESTER.

APPENDIX.

1.

STANDARD SPECIFICATIONS.

1.

Standard Specifications for Cement.

Adopted August 15th, 1908, by the
American Society for Testing Materials.

General Observations.

1. These remarks have been prepared with a view of pointing out the pertinent features of the various requirements and the precautions to be observed in the interpretation of the results of the tests.

2. The Committee would suggest that the acceptance or rejection under these specifications be based on tests made by an experienced person having the proper means for making the tests.

SPECIFIC GRAVITY.

3. Specific gravity is useful in detecting adulteration. The results of tests of specific gravity are not necessarily conclusive as an indication of the quality of a cement, but when in combination with the results of other tests may afford valuable indications.

FINENESS.

4. The sieves should be kept thoroughly dry.

TIME OF SETTING.

5. Great care should be exercised to maintain the test pieces under as uniform conditions as possible. A sudden change or wide range of temperature in the room in which the tests are made, a very dry or humid atmosphere, and other irregularities, vitally affect the rate of setting.

TENSILE STRENGTH.

6. Each consumer must fix the minimum requirements for tensile strength to suit his own conditions. They shall, however, be within the limits stated.

CONSTANCY OF VOLUME.

7. The tests for constancy of volume are divided into two classes, the first normal, the second accelerated. The latter should be regarded as a precautionary test only, and not infallible. So many conditions enter into the making and interpreting of it that it should be used with extreme care.

8. In making the pats the greatest care should be exercised to avoid

initial strains due to molding or to too rapid drying-out during the first twenty-four hours. The pats should be preserved under the most uniform conditions possible, and rapid changes of temperature should be avoided.

9. The failure to meet the requirements of the accelerated tests need not be sufficient cause for rejection. The cement may, however, be held for twenty-eight days, and a retest made at the end of that period using a new sample. Failure to meet the requirements at this time should be considered sufficient cause for rejection, although in the present state of our knowledge it cannot be said that such failure necessarily indicates unsoundness, nor can the cement be considered entirely satisfactory simply because it passes the tests.

SPECIFICATIONS.

General Conditions.

1. All cement shall be inspected.

2. Cement may be inspected either at the place of manufacture or on the work.

3. In order to allow ample time for inspecting and testing, the cement should be stored in a suitable weather-tight building having the floor properly blocked or raised from the ground.

4. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment.

5. Every facility shall be provided by the Contractor and a period of at least twelve days allowed for the inspection and necessary tests.

6. Cement shall be delivered in suitable packages with the brand and name of manufacturer plainly marked thereon.

7. A bag of cement shall contain 94 pounds of cement net. Each barrel of Portland cement shall contain 4 bags, and each barrel of natural cement shall contain 3 bags of the above net weight.

8. Cement failing to meet the seven-day requirements may be held awaiting the results of the twenty-eight day tests before rejection.

9. All tests shall be made in accordance with the methods proposed by the Committee on Uniform Tests of Cement of the American Society of Civil Engineers, presented to the Society January 21, 1903, and amended January 20, 1904, and January 15, 1908, with all subsequent amendments thereto. (See addendum to these specifications.)

10. The acceptance or rejection shall be based on the following requirements:

Natural Cement.

11. *Definition.* This term shall be applied to the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

FINENESS.

12. It shall leave by weight a residue of not more than 10 per cent. on the No. 100, and 30 per cent. on the No. 200 sieve.

TIME OF SETTING.

13. It shall not develop initial set in less than ten minutes, and shall not develop hard set in less than thirty minutes, or in more than three hours.

TENSILE STRENGTH.

14. The minimum requirements for tensile strength for briquettes one inch square in cross section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		50-100 lbs.
7 days (1 day in moist air, 6 days in water)		100-200 "
28 days (1 " " 27 " ")		200-300 "

One Part Cement. Three Parts Standard Sand.

7 days (1 day in moist air, 6 days in water)	25- 75 "
28 days (1 " " 27 " ")	75-150 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

15. Pats of neat cement about three inches in diameter, one-half inch thick at center, tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature.

(*b*) Another is kept in water maintained as near 70° F. as practicable.

16. These pats are observed at intervals for at least 28 days, and, to satisfactorily pass the tests, should remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

Portland Cement.

17. *Definition.* This term is applied to the finely pulverized product resulting from the calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials, and to which no addition greater than 3 per cent. has been made subsequent to calcination.

*For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 50 and 100 pounds, and so on for each period stated.

SPECIFIC GRAVITY.

18. The specific gravity of the cement, ignited at a low red heat, shall be not less than 3.10, and the cement shall not show a loss on ignition of more than 4 per cent.

FINENESS.

19. It shall leave by weight a residue of not more than 8 per cent. on the No. 100, and not more than 25 per cent. on the No. 200 sieve.

TIME OF SETTING.

20. It shall not develop initial set in less than thirty minutes, and must develop hard set in not less than one hour, nor more than ten hours.

TENSILE STRENGTH.

21. The minimum requirements for tensile strength for briquettes one inch square in section shall be within the following limits, and shall show no retrogression in strength within the periods specified.*

Age.	*Neat Cement.*	*Strength.*
24 hours in moist air		150-200 lbs.
7 days (1 day in moist air, 6 days in water)		450-550 "
28 days (1 " " 27 " ")		550-650 "

One Part Cement, Three Parts Standard Sand.

7 days (1 day in moist air, 6 days in water)	150-200 lbs.
28 days (1 " " 27 " ")	200-300 "

If the minimum strength is not specified, the mean of the above values shall be taken as the minimum strength required.

CONSTANCY OF VOLUME.

22. Pats of neat cement about three inches in diameter, one-half inch thick at the center, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*a*) A pat is then kept in air at normal temperature and observed at intervals for at least 28 days.

(*b*) Another pat is kept in water maintained as near 70° F. as practicable, and observed at intervals for at least 28 days.

(*c*) A third pat is exposed in any convenient way in an atmosphere of steam, above boiling water, in a loosely closed vessel for five hours.

23. These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, checking, cracking or disintegrating.

* For example, the minimum requirement for the twenty-four hour neat cement test should be some specified value within the limits of 150 and 200 pounds, and so on for each period stated.

SULPHURIC ACID AND MAGNESIA.

24. The cement shall not contain more than 1.75 per cent. of anhydrous sulphuric acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

2.

1. ABSTRACT EMBODYING PORTIONS APPLICABLE TO CONCRETE REINFORCEMENT FROM THE STANDARD SPECIFICATIONS FOR STRUCTURAL STEEL.

ADOPTED MARCH, 1906, by the
AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION.

Process of Manufacture. S3. Steel shall be made by the open-hearth process.

Schedule of Requirements. S4. The chemical and physical properties shall conform to the following limits:

Elements Considered.		Structural Steel.	Rivet Steel.
Phosphorus, max....	Basic....	0.04 per cent.	0.04 per cent.
	Acid..................	0.08 "	0.04 "
Sulphur, maximum.........................		0.05 "	0.04 "
Ultimate tensile strength Pounds per square inch.......................		Desired 60 000	Desired 50 000
Elongation, minimum, percentage in 8 in. Fig. 1		1 500 000* / Ult. tensile strength	1 500 000 / Ult. tensile strength
Character of fracture.............		Silky	Silky
Cold bends without fracture....		180° flat†	180° flat‡

* See paragraph 11. † See paragraphs 12, 13 and 14. ‡ See paragraph 15.

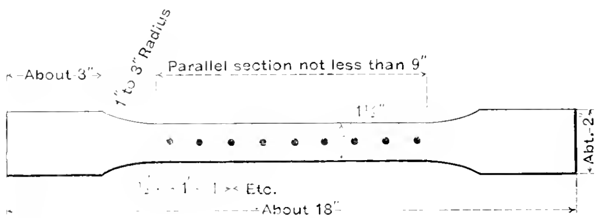


FIG. 1.

The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

Allowable Variations. S5. If the ultimate strength varies more than 4 000 lb. from that desired, a retest shall be made on the same gauge, which, to be acceptable, shall be within 5 000 lb. of the desired ultimate.

Chemical Analyses. S6. Chemical determinations of the percentages of carbon, phosphorus, sulphur, and manganese shall be made by the manufacturer

from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Check analyses shall be made from finished material, if called for by the purchaser, in which case an excess of 25% above the required limits will be allowed.

92. At least one tensile and one bending test shall be made from each melt of steel as rolled. Number of Tests.

95. Full-sized material for steel 1 in. thick and over, tested as rolled, shall bend, cold, 180° around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of bend. Thick Material.

98. Finished material shall be free from injurious seams, flaws, cracks, defective edges or other defects, and have a smooth, uniform and workmanlike finish. Plates 36 in. in width and under shall have rolled edges. Finish.

99. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Stamping.

100. Material which, subsequent to the above tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks, or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost. Defective Material.

3.

Alternative Specifications for Metal Reinforcement.

A.

1. For wire and rod reinforcements, the steel may be manufactured from Bessemer billets (not re-rolled rails) and shall meet the following requirements:

2. *Tensile Tests.*

Tensile strength, in pounds per square inch, not less than 105 000.

Yield point, in pounds per square inch, not less than 52 500 nor more than 90% of the tensile strength.

Elongation in 8 in., not less than 10%, with the following modifications:

a. For each increase in diameter of $\frac{1}{8}$ in. above $\frac{3}{4}$ in. a deduction of 1% shall be made from the specified elongation.

b. For materials from $\frac{1}{4}$ in. to, but not including, $\frac{5}{16}$ in. diameter the elongation shall be 8 per cent.

For material $\frac{3}{16}$ in. to $\frac{1}{4}$ in., elongation 7 per cent.

For material $\frac{1}{8}$ in. to $\frac{3}{16}$ in., elongation 6 per cent.

For material less than $\frac{1}{8}$ in., elongation 5 per cent.

3. *Bending Tests.*

Test specimens for bending shall be bent, cold, under the following conditions, without fracture on the outside of the bent portion:

Around twice their own diameter:	Around their own diameter:
1 in. diameter, 80 degrees.	¼ in. diameter, 130 degrees.
¾ in. diameter, 90 degrees.	$\frac{3}{16}$ in. diameter, 140 degrees.
½ in. diameter, 110 degrees.	⅛ in. diameter or less, 180 degrees.

B.

Steel wire used for reinforcement should be drawn from bars of basic open-hearth steel of the same quality as that specified for rivet steel.

Test pieces of wire shall bend 180° around their own diameter without fracture.

Suggested Formulas for Reinforced Concrete Construction.

These formulas are based upon the assumptions and principles given in the chapter on Design.

a. Standard Notation.

1. *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel.
f_c = compressive unit stress in concrete.
E_s = modulus of elasticity of steel.
E_c = modulus of elasticity of concrete.
$n = E_s \div E_c$.
M = moment of resistance, or bending moment in general.
A = steel area.
b = breadth of beam.
d = depth of beam to center of steel.
k = ratio of depth of neutral axis to effective depth, d.
z = depth of resultant compression below top.
j = ratio of lever arm of resisting couple to depth, d.
$jd = d - z$ = arm of resisting couple.
p = steel ratio (not percentage).

I-*Beams.*

b = width of flange.
b' = width of stem.
t = thickness of flange.

Beams Reinforced for Compression.

A' = area of compressive steel.
p' = steel ratio for compressive steel.

f_s' = unit compressive stress in steel.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C'.

Shear and Bond.

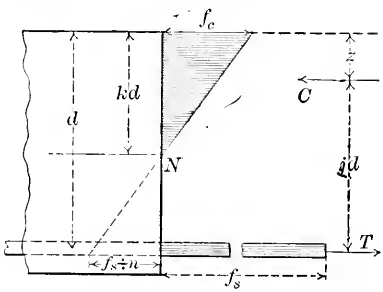
V = total shear.
v = shearing unit stress.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σ_o sum of the perimeters of all bars.

Columns.

A = total net area.
A_s = area of longitudinal steel.
A_c = area of concrete.
P = total safe load.

b. Formulas.

1. *Rectangular Beams.*



Position of neutral axis,

$$k = \sqrt{2\,p\,n + (p\,n)^2} - p\,n.$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}\,k.$$

[For $f_s = 15\,000$ to $16\,000$ and $f_c = 600$ to 650, j may be taken at $\frac{7}{8}$.]

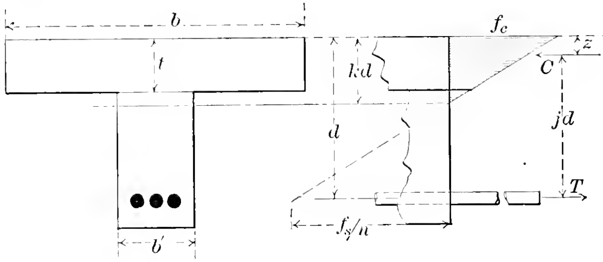
Fiber stresses,

$$f_s = \frac{M}{A\,j\,d} = \frac{M}{p\,j\,b\,d^2}. \quad f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k}.$$

Steel ratio,

$$p = \frac{1}{2}\,\frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{n\,f_c} + 1\right)}.$$

2. T-Beams.



Case I. When the neutral axis lies in the flange, use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem:

Position of neutral axis,

$$k\,d = \frac{2\,n\,d\,A + b\,t^2}{2\,n\,A + 2\,b\,t}.$$

Position of resultant compression,

$$z = \frac{3\,k\,d - 2\,t}{2\,k\,d - t} \cdot \frac{t}{3}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{M\,k\,d}{b\,t\,(k\,d - \frac{1}{2}\,t)\,j\,d} = \frac{f_s}{n} \cdot \frac{k}{k - 1}.$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem.

Position of neutral axis,

$$k\,d = \sqrt{\frac{2\,n\,d\,A + (b - b')\,t^2}{b'} + \left(\frac{n\,A + (b - b')\,t}{b'}\right)^2} - \frac{n\,A + (b - b')\,t}{b'}.$$

Position of resultant compression,

$$z = \frac{(k\,d\,t^2 - \frac{2}{3}\,t^3)\,b + [(k\,d - t)^2\,(t + \frac{1}{3})(k\,d - t)]\,b'}{t\,(2\,k\,d - t)\,b + (k\,d - t)^2\,b'}.$$

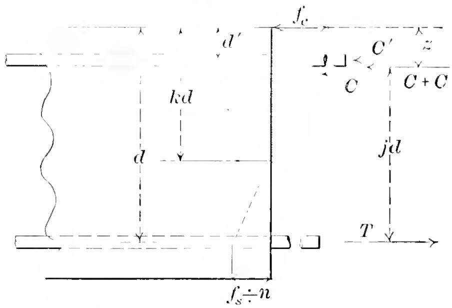
Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_s = \frac{M}{A\,j\,d}. \qquad f_c = \frac{2\,M\,k\,d}{[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b']\,j\,d}.$$

3. *Beams reinforced for compression.*



Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p').$$

Position of resultant compression.

$$z = \frac{\frac{1}{3}\,k^3\,d + 2\,p'\,n\,d'\left(k - \frac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \frac{d'}{d}\right)}.$$

Arm of resisting couple,

$$j\,d = d - z.$$

Fiber stresses,

$$f_c = \frac{6\,M\,k}{b\,d^2\left[3\,k - k^2 + \frac{6\,p'\,n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]}$$

$$f_s = \frac{M}{p\,j\,b\,d^2} = n\,f\,\frac{1 - k}{k}.$$

$$f_s' = n\,f_c\,\frac{k - \frac{d'}{d}}{k}.$$

4. *Shear, Bond, and Web Reinforcement.*

In the following formula, Σ_o, refers only to the bars constituting the tension reinforcement at the section in question and $j\,d$ is the lever arm of the resisting couple at the section.

For rectangular beams,

$$v = \frac{V}{b\,j\,d}.$$

$$u = \frac{V}{j\,d\,.\,\Sigma_o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

The stresses in web reinforcement may be estimated by using the following formulas:

Vertical reinforcement,

$$P = \frac{V s}{j d}.$$

Reinforcement inclined at 75°,

$$P = 0.7 \frac{V s}{j d}.$$

in which P = stress in single reinforcing member, V = proportion of total shear assumed as carried by the reinforcement, and s = horizontal spacing of the reinforcing members.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-beams,

$$v = \frac{V}{b' j d}, \quad u = \frac{V}{j d \cdot \Sigma_o}.$$

[For approximate results, j may be taken at $\frac{7}{8}$.]

5. *Columns.*

Total safe load,

$$P = f_c (A_c + n A_s) = f_c A (1 + (n - 1) p).$$

Unit stresses, $f_c = \dfrac{P}{A (1 + (n - 1) p)}$

$$f_s = n f_c.$$

MINORITY REPORT.

To the Members of the American Society of Civil Engineers:

The undersigned, members of the Special Committee on Concrete and Reinforced Concrete, who did not indorse the majority report of the Committee as submitted to the American Society of Civil Engineers, beg leave to present their reasons for dissenting in some very essential points from the views of the majority of the members of the Committee.

1. Form of Report.—We do not consider the report as presented to be in proper shape for a document to be submitted to a society composed of engineers. As we understand the duty of your Committee to be "to investigate and report", we are of the opinion that the report should consist of a dissertation on our investigations, with the conclusions derived therefrom. Some chapters of this report are more in the shape of specifications or arbitrary rules, such as an engineer might prepare for the use of his subordinates. We consider the report as a whole to be hastily prepared, and neither carefully considered, nor systematically arranged.

2. Design.—The chapter on design is limited to a description of minor details of a particular kind of construction, generally, and sometimes improperly, used in reinforced concrete work of buildings, to the exclusion of other types; to which limitations a designer with more advanced ideas, who does not follow traditions, would naturally object. As the art of reinforced concrete construction is yet in its infancy, the tentative designs and methods adopted by the pioneers in their attempts to utilize a new material cannot be expected to be perfect, but more or less crude and unscientific, as has been the case in structures built of other materials, which, by the natural process of evolution, have gradually developed into more rational forms.

Had the report been prepared at the time your Committee was appointed, about four years ago, when this so-called progress report could have been made just as well as now, it would probably have been considered as giving a fair résumé of the state of the art at that time. The progress made and the experience gained in the last four years have not been given sufficient attention, and we consider the recommendations referring to design not broad enough and not up to date.

3. Working Stresses.—Scientific facts are not established by a vote. They should be deduced from experimental data and practical experience, and should appeal to reason and judgment. The working strains on concrete, as recommended, are arbitrarily assumed, without reference to any tests, or facts established by experience, and without giving any reason for departing from the practice established by years

of experience. In other words, the members of the Society are asked to believe in the superior judgment of the members of the Committee who are responsible for this report.

The properties of reinforced concrete are not yet as well established as those of nearly all other building materials, owing to the lack of conclusive tests. No recognized theory for computing the internal forces in reinforced concrete is in existence at the present time, and as the data which are now available have not yet been sufficiently confirmed by practical experience, they should be used with the utmost caution, so that if there be an error, it will be on the side of safety. The number of disasters in concrete structures has not diminished, but rather increased; a word of warning, therefore, would be appropriate at this time.

We are of the opinion that the acceptance of the report in its present shape would have a tendency to encourage questionable practice, which is not in harmony with the aims and objects of this Society.

It may be said that the Society disclaims all responsibility for statements and opinions expressed in any of its publications; nevertheless, the statements embodied in a report of a Special Committee appointed to report on a special subject by the American Society of Civil Engineers, if printed in its *Transactions,* will be quoted as being indorsed by that Society.

The members forming the minority of the Special Committee on Concrete and Reinforced Concrete are of the opinion that the majority report should be referred back to the Committee for revision.

Respectfully submitted,

JANUARY, 1909.

C. C. SCHNEIDER, *Chairman,*
EMIL SWENSSON,
J. E. GREINER,
OLAF HOFF.

EXCURSIONS AND ENTERTAINMENTS AT THE FIFTY-SIXTH ANNUAL MEETING.

Wednesday, January 20th, 1909.—After the Business Meeting, lunch for over 700 members was served at 1 P. M., at the Society House.

At 3 P. M., by invitation of the Commissioner of the Department of Bridges and of the Commissioner of the Department of Docks and Ferries, there was an excursion to the Queensboro (Blackwell's Island) Bridge and the Williamsburg Bridge, and then to the Chelsea Section of Docks, North River. The party was conveyed in automobiles furnished through the courtesy of the Pennsylvania Steel Company and of Messrs. Snare and Triest and Messrs. R. P. and J. H. Staats. Those who could not be accommodated in the automobiles were taken by steamer, provided through the courtesy of the Commissioner of Charities, from the pier at the foot of East Sixtieth Street, down the East River, around the Battery and up the North River to the Chelsea Piers, where refreshments were served.

At 9 P. M., there was a reception, with dancing, in the Society House, at which the attendance was about 350.

Thursday, January 21st, 1909.—The day was devoted to an excursion to the Ashokan Reservoir by invitation of the Board of Water Supply. The party took the ferry at the foot of West Forty-second Street at 9 A. M., and the journey to Brown's Station was made by special train over the West Shore, and Ulster and Delaware Railroads, stops being made at Newburgh and Kingston.

The excursion afforded an excellent opportunity to view a number of historic points on the Hudson, and to inspect the basin of the Ashokan Reservoir and Olive Bridge Dam and the sites of the several dams and dikes, as well as the contractors' plant.

On the arrival of the party lunch was served at the contractors' camp by invitation of Messrs. MacArthur Bros. Co. and Winston and Co.

The train returning left Olive Bridge Dam at 3 P. M. and reached New York City at 6.30 P. M.

In the evening, at the Society House, there was a social and informal "Smoker," at which there was an attendance of more than 700.

The following list contains the names of 726 members, of various grades, who registered as being in attendance at the Annual Meeting. The list is incomplete, as some members failed to register, and it does not contain the names of any of the guests of the Society or of individual members. The number of guests is estimated at 400.

Adams, Arthur...New York City
Aiken, W. A....Philadelphia, Pa.
Aims, Walton I...New York City
Alderson, A. B., West Hartford, Conn.
Alexander, H. J..New York City

Allardice, E. R. B. .Clinton, Mass
Allen, C. Frank. . . .Boston, Mass.
Allen, Henry C. . .Syracuse, N. Y.
Allen, Kenneth. . .New York City
Andrews, Horace. . .Albany, N. Y.
Archbald, James. . . .Scranton, Pa.
Armstrong, R. S. .New York City
Ash, Dorsey. .San Francisco, Cal.
Atkinson, A.New Brunswick, N. J.
Atwood, T. C. . . . .Yonkers, N. Y.
Auryansen, F. . . . .Jamaica, N. Y.

Babcock, W. S. . . .New York City
Baird, H. C. . . . . .New York City
Baldwin, F. H. . . .Brooklyn, N. Y.
Ball, C. B. . . . . .Chicago, Ill.
Bance, C. W. . . .Jersey City, N. J.
Barker, J. M. . . . . . .Boston, Mass.
Barnes, M. G. . . . . .Albany, N. Y.
Barnes, T. H., Puerto Barrios, Guatemala
Barney, P. C. . . . .Brooklyn, N. Y.
Barrett, R. E. . . . .New York City
Barrows, H. K. . . . .Boston, Mass.
Barshell, F. B. . . .New York City
Bascom, H. F. . . . .Allentown, Pa.
Basinger, J. G. . . .New York City
Bates, Onward. . . . . . .Chicago, Ill.
Baucus, W. I. . . . . .Havana, Cuba
Baum, George. . . .Yonkers, N. Y.
Becker, E. J. . . .Waterford, N. Y.
Beer, Paul. . . . .Des Moines, Iowa
Belden, E. T. . . . .Pittsfield, Mass.
Belknap, J. M. . . .New York City
Belknap, W. E. . . .New York City
Bellinger, L. F. .Bath Beach, N. Y.
Belzner, T. . . . . . . .New York City
Bensel, J. A. . . . . .New York City
Benton, L. S. . . . .New York City
Benzenberg, G. H., Milwaukee, Wis.
Berger, Bernt. . . .New York City
Bernstein, L. . . . . .Baltimore, Md.
Bettes, C. R. .Far Rockaway, N. Y.
Betts, R. T. . . . . .Brooklyn, N. Y.
Beugler, E. J. . . . .New York City
Bieler, A. H. . . .Englewood, N. J.
Bissell, H. . . . .W. Medford, Mass.
Blakeslee, C. . .New Haven, Conn.
Blanchard, A. H., Providence, R. I.
Blanchard, M. . . . .New York City
Bland, J. C. . . . . . . .Pittsburg, Pa.
Blythe, L. H. . . .Philadelphia, Pa.
Boardman, H. E. .New York City
Bogert, C. L. . . . . .New York City
Boller, A. P. . .East Orange, N. J.
Boller, A. P., Jr. . .New York City
Bond, G. M. . . . . .Hartford, Conn.
Boniface, A. . . . . .Scarsdale, N. Y.
Booth, G. W. . . . . .New York City
Boucher, W. J. . . .New York City
Bower, C. P. . . .Philadelphia, Pa.
Bowman, A. L. . . .New York City
Boyd, J. C. . . . . . .Montclair, N. J.
Boyd, R. W. . . . . .New York City
Brackenridge, J. C., Richmond Hill, N. Y.
Brackenridge, W. A., Buffalo, N. Y.
Brackett, Dexter. . . .Boston, Mass.
Bradley, F. E. . . . .New York City
Branne, J. S. .Mt. Vernon, N. Y.
Braune, G. M. . . . . .Albany, N. Y.
Breckinridge, W. L. .Chicago, Ill.
Breed, C. B. . . . . . . .Boston, Mass.
Brendlinger, P. F., Philadelphia, Pa.
Brodie, O. L. . . . . .New York City
Brown, C. E., Hudson Heights, N. J
Brown, W. L. . . . .Montclair, N. J.
Brown, W. P. . . . .New York City
Burden, James. . . . . .Troy, N. Y.
Burgess, G. H. . . .New York City
Bush, E. W. . . . . .Hartford, Conn.
Bush, L. . . . . . .East Orange, N. J.

Caldwell, G. B. . . .Yonkers, N. Y.
Cantine, E. I. .East Orange, N. J.

Cantwell, H. E....New York City
Carpenter, A. W..Yonkers, N. Y.
Carpenter, C. E...Yonkers, N. Y.
Carr, Albert...East Orange, N. J.
Chadwick, C. R...New York City
Chambers, R. H...New York City
Chappell, T. F....New York City
Chase, J. C..Derry Village, N. H.
Chase, R. D..New Bedford, Mass.
Chase, W. H.....Flushing, N. Y.
Chester, J. N......Pittsburg, Pa.
Christian, G. L...New York City
Christie, J......Philadelphia, Pa.
Christy, G. L.....New York City
Church, E. C.....New York City
Churchill, C. S.....Roanoke, Va.
Clapp, O. F.....Providence, R. I.
Clark, G. H......New York City
Clarke, E. W......Babylon, N. Y.
Clarke, G. C......New York City
Clarke, St. John....Bogota, N. J.
Clement, F. H...Philadelphia, Pa.
Closson, E. S.....Brooklyn, N. Y.
Coburn, H. B., Jr.New York City
Codwise, E. B....Kingston, N. Y.
Cohen, F. W.....Harrisburg, Pa.
Cole, G. N.......New York City
Cole, H. J.....East Orange, N. J.
Cole, H. N......Providence, R. I.
Collier, B. C.......New York City
Collingwood, F...Elizabeth, N. J.
Conger, A. A......New York City
Conkling, C. C.....Buffalo, N. Y.
Cook, F. S........Yonkers, N. Y.
Cook, J. H.........Passaic, N. J.
Cook, J. W........Passaic, N. J.
Coombs, S. E.....Yonkers, N. Y.
Coomer, R. M....New York City
Cooper, S. L......Yonkers, N. Y.
Cornell, J. N. H..New York City
Coyne, H. L......New York City
Crandall, C. L......Ithaca, N. Y.
Crane, A. S......New York City
Crane, F. E....Amsterdam, N. Y.
Crane, J. S........Newark, N. J.
Crane, W. E.....Brooklyn, N. Y.
Cresson, B. F., Jr.New York City
Creuzbaur, R. W..Brooklyn, N. Y.
Crooks, C. H.....New York City
Crowell, Foster...New York City
Cuddeback, A. W..Paterson, N. J.
Cullen, J. F....Philadelphia, Pa.
Curtis, F. S........Boston, Mass.

Daggett, F. W......Boston, Mass.
Davis, A. L....Mt. Vernon, N. Y.
Davis, A. P....Washington, D. C.
Davis, C. E..Brown Station, N. Y.
Davis, J. L.......New York City
Deans, J. S......Phœnixville, Pa.
de Gratresse, J. R.New York City
Dennis, W. F..Washington, D. C.
Develin, R. G....Philadelphia, Pa.
Devin, George....New York City
Devlin, H. S....Brooklyn, N. Y.
de Wyrall, C., Ridgefield Park, N. J.
Deyo, S. L. F.....New York City
Dimon, D. Y.......Passaic, N. J.
Dodge, S. D., Cornwall-on-Hudson, N. Y.
Donham, B. C....New York City
Dorrance, W. T...New York City
Dougherty, R. E..New York City
Douglas, W. J..Washington, D. C.
Dunham, H. F....New York City

Earle, Thomas......Steelton, Pa.
Easterbrooks, P. B., Brooklyn, N. Y.
Eddy, H. P........Boston, Mass.
Edwards, J. H.....Passaic, N. J.
Ehrbar, L. H....New York City
Ellis, J. W.....Woonsocket, R. I.
Ely, C. B........Harrisburg, Pa.
Emerson, Guy C....Boston, Mass.
Emmons, C. M...Beaver Falls, Pa.
Engle, R. L..Johnson City, Tenn.
Erlandsen, O.....New York City
Esselstyn, H. H..Brooklyn, N. Y.

Evans, E. G....Hampton, Canada
Evans, J. M......Brooklyn, N. Y.
Everham, A. C.....Detroit, Mich.

Falk, M. S........New York City
Farley, J. M..White Plains, N. Y.
Farnham, R., Jr.,
Washington, D. C.
Fay, F. H..........Boston, Mass.
Fehr, H. R...........Easton, Pa.
Felton, H. C......Camden, N. J.
Fenton, L. G.....Brooklyn, N. Y.
Ferguson, G. R...New York City
Ferris, F. E.....Jersey City, N. J.
Fetherston, J. T.,
New Brighton, N. Y.
Firth, E. W......Jamaica, N. Y.
Fisher, E. A.....Rochester, N. Y.
Fisher, F. D.....Brooklyn, N. Y.
Fisher, H. T....Philadelphia, Pa.
Fitch, C. L.......Brooklyn, N. Y.
Fletcher, Robert..Hanover, N. H.
Flinn, A. D.......New York City
Ford, Harry C....New York City
Ford, W. G......Brooklyn, N. Y.
Fougner, H......New York City
Francis, G. B....New York City
Fraser, C. E.,
Richmond Hill, N. Y.
Frazee, J. H......New York City
Frazer, J. S......Brooklyn, N. Y.
French, A. H....Brookline, Mass.
French, A. W...Worcester, Mass.
French, J. B.....New York City
French, M. J........Utica, N. Y.
Frost, G. H......New York City
Frost, G. S......Brooklyn, N. Y.
Frye, A. I.......New York City
Fuller, F. L........Boston, Mass.
Fuller, G. W.....New York City
Fuller, W. B.....New York City
Fuller, W. E.....Brooklyn, N. Y.
Furber, W. C...Philadelphia, Pa.

Gahagan, W. H..Brooklyn, N. Y.
Gaiger, F. M.....New York City
Gandolfo, J. H....New York City
Gardiner, F. W....Yonkers, N. Y.
Gartensteig, C....New York City
Gaston, L. P.....Somerville, N. J.
Gerber, E..........Pittsburg, Pa.
Gifford, G. E.....New York City
Gildersleeve, A. C..New York City
Giles, Robert.....New York City
Gillen, W. J.Brown Station, N. Y.
Gillespie, R. H...New York City
Goad, C. E......Toronto, Canada
Goddard, H. W......Boston, Mass.
Goodnough, X. H....Boston, Mass.
Goodrich, E. P...New York City
Goodridge, J. W..Brooklyn, N. Y.
Goodsell, D. B....New York City
Gowen, C. S......Ossining, N. Y.
Grady, C. B..West Orange, N. J.
Granbery, J. H....New York City
Gray, J. H.........Orange, N. J.
Gray, William....New York City
Green, B. R....Washington, D. C.
Green, C. N......New York City
Green, F. M......New York City
Greene, Carleton..New York City
Greene, G. S., Jr..New York City
Gregory, C. E....Mt. Kisco, N. Y.
Greiner, J. E.....Baltimore, Md.
Grimes, E. L.........Troy, N. Y.
Grimm, C. R....Brooklyn, N. Y.
Gross, C. A....So. Bethlehem, Pa.
Grossart, L. J. H..Allentown, Pa.
Grover, N. C....New York City
Gustin, R. P...Mt. Vernon, N. Y.
Guthrie, K. O....Brooklyn, N. Y.

Hague, C. A.....New York City
Haight, S. S.....New York City
Haines, C. W....Philadelphia, Pa.
Haines, E. G.,
Richmond Hill, N. Y.
Hale, H. M.......New York City
Hall, M. W.......New York City
Hallock, J. C......Newark, N. J.
Hancock, R. R....New York City

Hansel, Charles...New York City
Haring, A........New York City
Harlow, G. R......Pittsburg, Pa.
Harrington, F. F....Norfolk, Va.
Harrison, E. W..Jersey City, N. J.
Harte, C. R...West Haven, Conn.
Harwi, S. J.......Bayonne, N. J.
Haskins, W. J....New York City
Hauck, W........Brewster, N. Y.
Haviland, A......New York City
Hawley, C. R......Buffalo, N. Y.
Hayes, H. W.......Boston, Mass.
Hayes, S. W.......Geneva, N. Y.
Hazard, E.....Mauch Chunk, Pa.
Hazard, S....White Plains, N. Y.
Hazelton, C. W., Turners Falls, Mass.
Hazen, Allen.....New York City
Hazen, W. N.....New York City
Healy, J. R......New York City
Hedden, E. B., White Plains, N. Y.
Heller, J. W..West Orange, N. J.
Hench, N. M......Pittsburg, Pa.
Henderson, J. T..Hartford, Conn.
Herbert, H. M.Bound Brook, N. J.
Hering, R.......Montclair, N. J.
Herrick, J. J....Flushing, N. Y.
Herschel, C......New York City
Hewes, V. H.....New York City
Higgins, C. H..Jersey City, N. J.
Higgins, J. W., Roselle Park, N. J.
Higginson, J. Y., New Rochelle, N. Y.
Hildreth, J. L., Jr., Babylon, N. Y.
Hill, W. R.........Albany, N. Y.
Hoag, S. W., Jr..New York City
Hodgdon, F. W.....Boston, Mass.
Hoff, Olaf......Tarrytown, N. Y.
Hogan, J. P....High Falls, N. Y.
Holbrook, P......New York City
Holden, C. A.....Hanover, N. H.
Holmes, G. D....Syracuse, N. Y.
Holtsmark, E....Brooklyn, N. Y.
Honness, G. G., Pleasantville, N. Y.
Hood, J. N..Richmond Hill, N. Y.
Horan, J. J.....Brooklyn, N. Y.
Horne, Harold W., Cornwall-on-Hudson, N. Y.
Horton, H. E.......Chicago, Ill.
Hovey, O. E.....Plainfield, N. J.
Howard, J. L......Melrose, Mass.
Howell, W. A......Newark, N. J.
Hoyt, J. C.....Washington, D. C.
Hubbell, G. S....Flushing, N. Y.
Hudson, C. W....New York City
Huestis, C. C......Buffalo, N. Y.
Hughes, H. J...Cambridge, Mass.
Hulse, S. C.........Bedford, Pa.
Humphrey, R. L., Philadelphia, Pa.
Hunt, C. E.......New York City
Hunt, Charles Warren, New York City
Huntington, G. D..Massena, N. Y.
Hurst, G. J......New York City

Ingersoll, C. M., Jr., New York City
Irwin, J. C......New York City
Ives, A. S....Poughkeepsie, N. Y.

Janes, G. P........Roselle, N. J.
Japp, Henry....Richmond, N. Y.
Jenkins, J. E.....Medina, N. Y.
Jewel, L. L........Pittsburg, Pa.
Johannesson, S...New York City
Johnson, L. J...Cambridge, Mass.
Johnson, T. H.....Pittsburg, Pa.
Johnson, W. S.....Boston, Mass.
Jones, Henry L...New York City
Jones, I. P......Georgetown, Del.
Jonson, E. F......New York City
Jordan, E. C.......Portland, Me.
Jouett, H. D......New York City
Judson, W. P......Oswego, N. Y.
Just, G. A.......Scarsdale, N. Y.

Kastl, A. E. . . . . . . Peekskill, N. Y.
Keith, H. C. . . . . . New York City
Keller, O. B. . . . . . New York City
Kelley, W. D. . . . . . Yonkers, N. Y.
Khuen, R. . . . . Pittsburg, Pa.
Killam, C. W. . . Cambridge, Mass.
Kimball, G. A. . . . . . Boston, Mass.
King, C. C. . New Brighton, N. Y.
King, H. W. . . . . . Cleveland, Ohio
King, P. S. . . . . . Philadelphia, Pa.
King, W., Jr. . . . . . New York City
Kirkwood, H. C. . New York City
Klapp, Eugene. . . . Wyckoff, N. J.
Knap, E. D. . . . . . . Yonkers, N. Y.
Knap, Joseph M. . New York City
Knickerbocker, C. E., Middletown, N. Y.
Knighton, J. A. . . New York City
Kolb, H. J. . . . . . Brooklyn, N. Y.
Kornfeld, A. E. . . New York City
Kroeber, A. T. . . . . . Reading, Pa.

La Chicotte, H. A. New York City
Landers, C. S. . . . New York City
Langthorn, J. S. . Kingston, N. Y.
Larsson, C. G. E. . Plainfield, N. J.
Lathrop, J. C. . Jersey City, N. J.
Lavis, F. . . . . . . Mt. Vernon, N. Y.
Lea, R. S. . . . . . Montreal, Canada
Leather, B. H. . . . New York City
Lee, W. B. . . . . Hillburn, N. Y.
Leffingwell, F. D., Upper Montclair, N. J.
Lentilhon, E. . . . Bay Shore, N. Y.
Leonard, C. M. . . . . . . Chicago, Ill.
Leonard, H. R. . Philadelphia, Pa.
Lesley, R. W. . . Philadelphia, Pa.
Lewis, E. W. . . New Haven, Conn.
Lewis, N. P. . . . . . New York City
Lex, W. I. . . . . . . Philadelphia, Pa.
Lindenthal, G. . . . . New York City
Llewellyn, F. J. . . . . . Chicago, Ill.
Llewellyn, F. T. . . New York City
Lobo, C. . . . . Brooklyn, N. Y.
Lockwood, J. A. . . New York City
Lockwood, W. D., Schenectady, N. Y.
Logan, W. S. . . . Arlington, N. J.
Long, E. McL. . . . New York City
Look, M. J. . Brown Station, N. Y.
Loomis, H. . . . . Mt. Vernon, N. Y.
Low, G. E. . . . . New York City
Loweth, C. F. . . . . Chicago, Ill.
Lundie, J. . . . . New York City
Lundoff, C. W. . . . Cleveland, Ohio
Luster, W. H., Jr., Elizabeth, N. J.
Lynn, H. H. E. . . . New York City

MacCracken, G. G., New York City
Macdonald, C., Gananoque, Canada
MacGregor, R. A. . New York City
Machen, H. B. . . . New York City
Macksey, H. V. . . . . Boston, Mass.
McBurney, H. West Haven, Conn.
McCarthy, G. A., North Bay, Canada
McClave, S. W., Jr., Grantwood, N. J.
McCord, J. B. . . . . Elizabeth, N. J.
McCurdy, H. S. R., Brown Station, N. Y.
McInnes, F. A. . . . . . Boston, Mass.
McIntire, T. B. . . . Yonkers, N. Y.
McKenzie, T. H., Southington, Conn.
McLure, N. R. . . . Phœnixville, Pa.
McMinn, T. J. . . . . New York City
McNaugher, D. W. . Pittsburg, Pa.
McNeal, J. . . . . Easton, Pa.
McPherson, R. H. . New York City
Madden, J. H. . . . . New York City
Maier, H. L. . . . . Wilmington, Del.
Maignen, J. P. A., Philadelphia, Pa.
Mair, J. W. . . . Mt. Vernon, N. Y.
Manley, Henry. . . . . Boston, Mass.
Mansfield, R. J. . . New York City
Marburg, E. . . . . Philadelphia, Pa.
Marden, H. H., Jr. New York City

Marden, W. R. . . . . . Albany, N. Y.
Marple, W. McK. . . Scranton, Pa.
Matheson, J. D. Winnipeg, Canada
Mayell, A. J. . . . . . New York City
Mead, C. A.,
Upper Montclair, N. J.
Meadowcroft, W. . New York City
Mebus, C. F. . . . Philadelphia, Pa.
Meem, J. C. . . . . . Brooklyn, N. Y.
Meggy, R. L. G. . Plainfield, N. J.
Mehren, E. J. . . . . New York City
Melius, L. L. . . . . . New York City
Merrill, Ogden. . . . New York City
Merriman, T. . . . . . New York City
Merritt, D. S. . . . Tarrytown, N. Y.
Merryman, W. C. . New York City
Metcalf, Leonard. . . Boston, Mass.
Meyers, C. W. . . . . New York City
Miller, Frank. . . . New York City
Miller, H. C. . . . . . New York City
Miller, H. A.,
Newton Highlands, Mass.
Miller, R. P. . . . . . New York City
Mills, C. M. . . . . Philadelphia, Pa.
Minor, E. E. . . New Haven, Conn.
Mitchell, S. P. . Philadelphia, Pa.
Modjeski, Ralph. . . . . Chicago, Ill.
Moisseiff, L. S. . . . New York City
Molitor, David A. . . . Ithaca, N. Y.
Molitor, F. . . . . . Philadelphia, Pa.
Monahan, J. J.,
West Chelmsford, Mass.
Moncure, W. A. . Philadelphia, Pa.
Montfort, R. . . . . . . Louisville, Ky.
Moore, W. H. . New Haven, Conn.
Morse, G. F. . . . . . Bayonne, N. J.
Morton, W. S. . . . New York City
Moulton, Guy. . . . Syracuse, N. Y.
Moulton, Mace. . . New York City
Musson, E. F. . . . . Norwich, N. Y.
Myers, C. H. . . . . . New York City
Myers, J. H. . White Plains, N. Y.

Neale, John C. . . . . . Pittsburg, Pa.
Nelson, J. C. . . . . . New York City
Nethercut, E. S. . . . . Chicago, Ill.
Neumeyer, R. E. . Bethlehem, Pa.
Nial, W. A. . . . . . . New York City
Nichol, H. S. . . . . . New York City.
Nichols, A. R. . Washington, D. C.
Nichols, C. H. . New Haven, Conn.
Noble, Alfred. . . . . New York City
North, Edward P. . New York City
Northrop, A. A.,
Brown Station, N. Y.
Nye, A. S. . . . . New York City

Oakley, G. I. . . Little Falls, N. Y.
Obreiter, J. W. . . . Hoboken, N. J.
O'Brien, J. H. . . . New York City
Odell, F. S. . . Port Chester, N. Y.
Ogden, Henry N. . . . Ithaca, N. Y.
Olmsted, A. E. . . . New York City
Opdyke, S. B., Jr. . New York City
O'Rourke, J. F. . . . New York City
Orr, Alexander. Gloversville, N. Y.
Orrok, G. A. . . . . . Brooklyn, N. Y.
Osborn, F. C. . . . . Cleveland, Ohio
Osgood, J. O. . . Jersey City, N. J.
Ostrup, J. C. . . . . . New York City
Ott, S. J. . . . . . . Rutherford, N. J.
Owen, James. . . . Montclair, N. J.
Owen, K. D. . . . . Montclair, N. J.
Oxholm, T. S.,
West New Brighton, N. Y.

Parker, C. J. . . . . New York City
Parker, William. . . . Boston, Mass.
Parks, O. E. . . . . . Westfield, Mass.
Parmley, W. C.,
Upper Montclair, N. J.
Parsons, G. W. . . . . . . Steelton, Pa.
Parsons, H. A. . . Stamford, Conn.
Parsons, H. de B. . New York City
Parsons, W. B. . . . New York City
Pegram, G. H. . . . New York City
Pendlebury, E. . . . Arlington, N. J.
Perkins, P. S. . . . Providence, R. I.
Perry, J. P. H. . . . New York City
Peterson, John. . . . New York City

Pflueger, A. C....New York City
Phelps, W. C.....New York City
Philips, J. H.......Newark, N. J.
Phillips, Asa E. Washington, D. C.
Pike, R. A.....Mt. Vernon, N. Y.
Plogsted, W. J....New York City
Pohl, C. A........New York City
Polk, W. A.......New York City
Pollock, C. D.....New York City
Poole, C. A.....Waterford, N. Y.
Porter, J. E......Yonkers, N. Y.
Post, Henry W., Richmond Hill, N. Y.
Potter, A.........New York City
Potts, Clyde......New York City
Pratt, A. H......New York City
Pratt, W. A.......New York City
Price, P. L....East Orange, N. J.
Priest, B. B...East Orange, N. J.
Pruyn, F. L......New York City
Pugh, M. R.....Philadelphia, Pa.
Putnam, G. R..Washington, D. C.

Quick, A. M......Baltimore, Md.
Quincy, C. F.....New York City

Read, R. L........Malden, Mass.
Reid, H. A...Dobbs Ferry, N. Y.
Rennell, H. H..Bridgeport, Conn.
Reynders, J. V. W...Steelton, Pa.
Rice, G. S........New York City
Richardson, C....New York City
Richardson, J. D...Corona, N. Y.
Richardson, J. H...Orange, N. J.
Ridgway, R..Poughkeepsie, N. Y.
Riggs, M. J.........Toledo, Ohio
Rights, L. D......New York City
Ripley, H. L....Providence, R. I.
Ripley, J..........Albany, N. Y.
Robb, L. A.........Newark, N. J.
Robbins, D. W....New York City
Robbins, H. R....New York City
Roberts, P., Jr..Philadelphia, Pa.
Robinson, A. W., Montreal, Canada
Robinson, E. W...New York City
Robinson, R. T. Weehawken, N. J.
Rogers, H. L.....Brooklyn, N. Y.
Rohrer, J. B.......Lancaster, Pa.
Rose, R. V..Niagara Falls, N. Y.
Ruggles, W. B..Cambridge, Mass.
Ryan, M. H......New York City
Ryder, E. M. T...New York City

Sabin, A. H......Flushing, N. Y
Sanford, H. C....New York City
Sass, C. W.......New York City
Saunders, G. C...New York City
Sayers, E. L......New York City
Sayles, R. W......New York City
Schaeffer, Amos...New York City
Schaeffler, J. C.....Boston, Mass.
Schall, F. E., South Bethlehem, Pa.
Schmidt, Max E...Convent, N. J.
Schneider, C. C., Philadelphia, Pa.
Schwarze, C. T., South Orange, N. J.
Schwitzer, J. E., Winnipeg, Canada
Scorgie, J. C...Cambridge, Mass.
Seaman, H. B....New York City
Searle, C. D......New York City
Searle, L. F., Brown Station, N. Y.
Senior, F. S...Montgomery, N. Y.
Shaw, D. J.....Chappaqua, N. Y.
Shellenberger, L. R., Bayonne, N. J.
Sherrerd, J. M.......Easton, Pa.
Sherrerd, M. R.....Newark, N. J.
Shoemaker, M. N...Newark, N. J.
Simpson, G. F....New York City
Sitt, W. T........New York City
Skillin, E. S....Glen Ridge, N. J.
Skinner, F. W., Tompkinsville, N. Y.
Skinner, J. F....Rochester, N. Y.
Slocum, C. L..New Haven, Conn.

Smith, E. G.......New York City
Smith, F. C.,
Great Barrington, Mass.
Smith, H. S....Wilkes-Barre, Pa.
Smith, J. Waldo..New York City
Smith, M. H..White Plains, N. Y.
Smith, T. G.......Buffalo, N. Y.
Smith, W. M., Sr.,
West New Brighton, N. Y.
Snow, J. B..Richmond Hill, N. Y.
Snyder, C. H......Oswego, N. Y.
Soper, G. A......New York City
Souther, T. W..West Point, N. Y.
Sparrow, W. W. K.,
Kansas City, Mo.
Spear, P. H........Raritan, N. J.
Spear, W. E.......Babylon, N. Y.
Spencer, W. T..New Haven, Conn.
Sperry, H. M.....New York City
Splitstone, C. H.,
East Orange, N. J.
Spofford, C. M....New York City
Spooner, A. W....New York City
Sprague, E. L., Jr.,
Valhalla, N. Y.
Stanford, H. R..Philadelphia, Pa.
Staniford, C. W..Brooklyn, N. Y.
Stanton, R. B.....New York City
Starr, H. H.....Philadelphia, Pa.
Stearns, F. L....Scarsdale, N. Y.
Stearns, F. P.......Boston, Mass.
Stearns, R. H.....New York City
Stern, E. W......New York City
Stevenson, W. F..New York City
Stewart, W. J....Rochester, N. Y.
Stickney, G. F.....Albany, N. Y
Stilson, M. A. O.,
Waterbury, Conn.
Stoddard, G. C....New York City
Stoddard, R. F....Milford, Conn.
Stone, E. E.........Boston, Mass
Stone, W. W.......Walden, N. Y.
Storey, W. B., Jr....Chicago, Ill.
Stowell, C. F.......Albany, N. Y.
Stowitts, G. P....New York City
Strobel, C. L........Chicago, Ill
Strouse, W. F.....Baltimore, Md.
Stuart, Francis Lee,
Essex Fells, N. J.
Suter, R.........New York City
Sutton, F......Washington, D. C
Swain, G. F........Boston, Mass.
Swanker, J. E..Schenectady, N. Y.
Swensson, E.......Pittsburg, Pa.
Swift, W. E.,
Cornwall-on-Hudson, N. Y.
Swindells, J. S....Jamaica, N. Y.

Taber, G. A......Brooklyn, N. Y
Tait, Harold......New York City
Tait, J. G........Metuchen, N. J.
Talbot, A. N.........Urbana, Ill.
Tallman, P. B..East Orange, N. J.
Tarr, C. W........Babylon, N. Y.
Taylor, A. J....Wilmington, Del.
Taylor, C. F......New York City
Taylor, E. A.....Uxbridge, Mass.
Taylor, L. A........Boston, Mass
Taylor, S. A.......Pittsburg, Pa.
Tenney, G. O......Richmond, Va.
Terry, J. H.....Philadelphia, Pa.
Theban, J. G.....New York City
Thomas, C. D....Brooklyn, N. Y.
Thomes, E. H.....Jamaica, N. Y.
Thompson, F.......Boston, Mass.
Thompson, S. E.,
Newton Highlands, Mass.
Thomson, T. K...New York City
Thornley, J........Albany, N. Y.
Tillson, G. W....Brooklyn, N. Y.
Tingley, R. H....New York City
Tinkham, S. E......Boston, Mass.
Titsworth, R. B..Plainfield, N. J.
Tompkins, E. de V..Pelham, N. Y.
Tórrey, J. E......Paterson, N. J.
Travell, W. B....Plainfield, N. J.
Tretter, G. A.......Roanoke, Va.
Tribus, L. L......New York City
Triest, W. G......New York City
Trout, C. E.......New York City

True, A. O.......New York City
Tull, R. W.......New York City
Turner, D. L.....New York City
Turner, E. K.......Boston, Mass.
Turner, H. C.....New York City
Turner, W. T.........Lima, Peru
Tuttle, A. S......New York City

Van Cleve, A. H., Niagara Falls, N. Y.
Van Horne, J. G..New York City
Vanneman, C. R....Albany, N. Y.
Van Norden, E. M., Brooklyn, N. Y.
Van Suetendael, A. O., Albany, N. Y.
Verrill, G. E..New Haven, Conn.
Vier, H......White Plains, N. Y
Vogleson, J. A..Philadelphia, Pa.
Volck, A. G.....Glen Cove, N. Y.
Voorhees, P.......Harrisburg, Pa
Vorce, C. B.......New York City

Waddell, J. A. L., Kansas City, Mo.
Waddell, N. E., Vancouver, Canada
Wadsworth, J. E..New York City
Wagner, B. M....Brooklyn, N. Y.
Wagner, H. E.....Andover, N. J.
Wagner, J. C....Philadelphia, Pa.
Waldron, S. P..East Orange, N. J
Walker, J. W......Pittsburg, Pa.
Ward, C. D.......New York City
Warren, H. A....New York City
Wason, L. C.....Brookline, Mass.
Wassner, M.......New York City
Watkins, F. W., White Plains, N. Y.
Watt, J. R.........Albany, N. Y.
Webster, A. L.....New York City
Webster, G. S...Philadelphia, Pa.
Wegmann, E......Yonkers, N. Y
Wells, C. E..White Plains, N. Y.
Wendt, E. F.......Pittsburg, Pa.
West, O. J..........Chicago, Ill.
Weston, R. S.......Boston, Mass.
Wheelock, De F. A...Warren, Pa.
Whinery, S....East Orange, N. J.
Whipple, G. C....New York City
Whiskeman, J. P..New York City
White, T. S.....Beaver Falls, Pa
White, W. M.....New York City
Whittemore, W. F..Hoboken, N. J.
Wiggin, E. W..New Haven, Conn.
Wiggin, T. H., White Plains, N. Y
Wilcock, F......Brooklyn, N. Y.
Wild, H. J.....Weehawken, N. J
Wiley, W. H.....New York City
Wilkes, J. K..New Rochelle, N. Y.
Wilkins, W. G......Pittsburg, Pa.
Williams, G. S..Ann Arbor, Mich.
Williamson, F. S..New York City
Wilson, C. W. S., New Rochelle, N. Y.
Wilson, H. M..Washington, D. C.
Wilson, P. H....Philadelphia, Pa.
Wilson, W. W.....New York City
Winsor, F. E..White Plains, N. Y.
Wise, C. R.........Passaic, N. J.
Wölfel, P. L.......Pittsburg, Pa.
Wolverton, I. M., Mt. Vernon, Ohio
Wood, H. B.......Belmont, Mass.
Woodard, S. H...Scarsdale, N. Y.
Woolley, A. F., Sylvan Beach, N. Y.
Worcester, J. R.....Boston, Mass.
Wormser, M.....Montclair, N. J.
Wortendyke N. D. Jersey City, N. J.
Worthington, C...New York City
Wrenn, J. F......New York City
Wyckoff, C. R., Jr., Brooklyn, N. Y.
Wyman, A. M..East Orange, N. J.

Yates, J. J......Plainfield, N. J.
Young, C. G......New York City

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, March 3d, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Action of Frost on Cement and Cement Mortar, Together with Other Experiments on These Materials," by Messrs. Ernest R. Matthews, and James Watson; and "The Bonding of New to Old Concrete," by E. P. Goodrich, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for January, 1909.

Wednesday, March 17th, 1909.—8.30 P. M.—A paper entitled "Steel Sheeting and Sheet-Piling," by L. R. Gifford, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, April 7th, 1909.—8.30 P. M.—A paper on "The Sixth Street Viaduct of Kansas City," by E. E. Howard, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, April 21st, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, M. Am. Soc. C. E.; and "Sampittic Surfacing," by W. W. Crosby, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION.

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE.

H. W. Hayes,

A. W. Dean, S. E. Tinkham,

H. D. Woods, J. F. Stevens,

George A. Kimball, J. W. Ellis.

PAPERS AND DISCUSSIONS

The Board of Direction has decided to issue *Transactions* in future as a quarterly publication. There will therefore be four volumes each year instead of two as heretofore.

There seems to be some misunderstanding about this matter, several members having protested on the ground that the present volumes are not too large, and that making them smaller would make reference to them more difficult, as well as increase the cost of binding.

It seems, therefore, necessary to explain that the decision of the Board is based on the fact that the amount of valuable material offered the Society for publication has so increased of late that a large part of it would have to be refused if only two volumes per annum were issued, and on the belief that the most effective way of increasing the usefulness of the Society to its Members and the Engineering Profession is to increase the volume of its Publications.

The decision was arrived at only after most careful consideration of the matter from every point of view, and it is hoped the result will be satisfactory to the membership.

It is also hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of the paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each must be established. It is expected that the first volume for 1909 will be issued on or about March 31st, the second on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur= und Architekten=Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur= und Architekten=Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineeer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

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* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From January 13th to February 9th, 1909)

DONATIONS*

HYDRAULIC TABLES.

The Elements of Gagings and the Friction of Water Flowing in Pipes, Aqueducts, Sewers, etc., as Determined by the Hazen and Williams Formula and the Flow of Water over Sharp-Edged and Irregular Weirs, and the Quantity Discharged as Determined by Bazin's Formula and Experimental Investigations upon Large Models. By Gardner S. Williams and Allen Hazen, Members, Am. Soc. C. E. Second Edition, Revised and Enlarged. Cloth, 9 x 6 in., illus., 6 + 104 pp. New York, John Wiley & Sons, 1909. $1.50.

In this edition all errors in the original text and tables, developed by the continuous use of the book for over three years are stated to have been corrected. The preface also states that, the changes in this edition are nearly all confined to that part of the book devoted to the flow of water over weirs, where some new matter on submerged weirs is presented in the text and the table of discharge by Bazin's formula has been extended to cover variations of head by 0.01 ft. from zero to 6 ft. A table of discharge of high weirs has also been added and these additions, with a new and more correct title-page and an extended table of contents, are intended to increase materially the usefulness of the work. The Contents are: Introduction; Formula Used; Increasing Friction with Age, How Computed and Indicated in the Tables; Observations of Flow in Cast-Iron Pipe; Observations of Flow in Riveted Steel Pipe; Observations of Flow in Wooden-Stave Pipe; Observations of Flow in Rectangular Wooden Pipe; Observations of Flow in Cement Pipe; Observations of Flow in Wrought-Iron Pipe; Observations of Flow in Galvanized-Iron Pipe; Observations of Flow in Brass Pipe; Observations of Flow in Lead Pipe; Observations of Flow in Glass Pipe; Observations of Flow in Fire-Hose; Observations of Flow in Open Conduits; Observations of Flow in Aqueducts; Observations of Flow in Brick Sewers; Observations of Flow in Canals; Table of Flow in Small Brass Pipes; Table of Flow in Wrought-Iron Pipes; Table of Flow in Hose and Pipes; Table of Flow in Pipes, 4 to 144 in. Diameter. Relative Discharging Capacity of Aqueducts. Table of Flow in Aqueducts; Sewers, Table of Slopes Required to Produce Certain Velocities; Tile Sewer Table; Circular Brick Sewer Table; Decrease in Carrying Capacity of Cast-Iron Pipe with Age; Comparison of Results with those of Coffin and Weston; Metric Pipe Table. Loss of Head in Venturi Meter; Underdrains, for Sand-Filters; Flow Over Weirs; Sharp-Edged Weirs, Discussion; Sharp-Edged Weirs, Tables of Discharge; Low Heads, and Contractions and Very High Weirs, Discussion; High Heads, Table of Discharge; High Weirs, Table of Discharge; Flat Crest and Other Weirs, Discussion; Submerged Weirs, Discussion; Multipliers for Flat Crest Weirs; Multipliers for Trapezoidal Weirs; Multipliers for Triangular Weirs; Multipliers for Compound Weirs; Multipliers for Complex Weirs.

ARTIFICIAL WATERWAYS AND COMMERCIAL DEVELOPMENT.

(With a History of the Erie Canal.) By A. Barton Hepburn. Cloth, 7½ x 5 in., 7 + 115 pp. New York, The Macmillan Company, 1909. $1.00.

The purpose of this volume, as declared by the author, is to place before the public, in concise form, the salient facts as to artificial waterways and their relation to commercial development. A general review of the canal systems of the world is given, with the history of the Erie Canal as typical of all. The Contents are: The World's Canals; The Canal System of New York, I, The Period of Inception, II, The Period of Development, III, The Present Conditions, IV, The Competition against New York City; The Panama Canal; The Waterways Question and Conservation of Our Resources; Appendix (Statistical Tables) *A*, Relating to New York Canals, *B*, Relating to Commerce of New York City, *C*, Miscellaneous; Index.

APPLIED MECHANICS FOR ENGINEERS.

A Text-Book for Engineering Students. By E. L. Hancock. Cloth, 7½ x 5 in., illus., 11 + 385 pp. New York, The Macmillan Company, 1909. $2.00 net.

This work is intended as a text-book for engineering students of the Junior year, and the subject-matter, except that contained in a few chapters, is stated to be

*Unless otherwise specified, books in this list have been donated by the publisher.

such as is usually covered in one semester. Each new principle developed is followed, it is stated, by a number of applications and each problem deals with matters that directly concern the engineer, and constitute practical engineering work. The Chapter headings are: Definitions; Concurrent Forces; Parallel Forces; Center of Gravity; Couples; Non-concurrent Forces; Moment of Inertia; Flexible Cords; Rectilinear Motion; Curvilinear Motion; Rotary Motion; Dynamics of Machinery; Work and Energy; Friction; Impact; Appendix I, Hyperbolic Functions, Tables; Appendix II, Logarithms of Numbers; Appendix III, Trigonometric Functions, Tables; Appendix IV, Squares, Cubes, Square Roots, etc., of Numbers; Appendix V, Conversion Tables; Index.

TIMBER.

By J. R. Baterden. Cloth, $8\frac{1}{2} \times 5\frac{1}{2}$ in., illus., 10 + 351 pp. London, Archibald Constable & Co., Ltd., 1908. 6 shillings net.

The author states that this book is intended to be essentially a practical work, with botany only incidentally touched upon. Only those timbers most generally used, either in their native districts, or in the general timber trade, are discussed, together with some likely to come into the market before long. Those timbers most used have been dealt with at greatest length. No information is given, the preface states, of which the author is not either certain from his own knowledge or has not gained from authentic sources. There is an appendix, giving sample cargoes of timber and showing the great variety and large quantities of timber landed in Great Britain, and a bibliography of one and one-half pages. The Contents are: Timber; The World's Forest Supply; Quantities of Timber Used; Timber Imports into Great Britain; European Timber; Timber of United States and Canada; Timbers of South America, Central America, and West India Islands; Timbers of India, Burma, and Andaman Islands; Timber of the Straits Settlements, Malay Peninsula, Japan, and South and West Africa; Australian Timbers; Timbers of New Zealand and Tasmania; Causes of Decay and Destruction of Timber; Seasoning and Impregnation of Timber; Defects in Timber, and General Notes; Strength and Testing of Timber; "Figure" in Timber; Appendix; Bibliography; Index.

Gifts have also been received from the following:

Am. Inst. of Elec. Engrs. 1 bound vol.
Am. Mathematical Soc. 1 pam.
Am. Soc. for Testing Materials. 1 vol.
Am. Soc. Mech. Engrs. 1 bound vol.
Architectural Inst. of Canada. 1 pam.
Atlantic City, N. J.-Water Dept. 2 pam.
Belgium-Ministère des Chemins de Fer Postes et Télégraphes. 1 vol.
Bengal, India-Irrig. Dept. 1 pam.
Bombay-Public Works Dept. 1 bound vol.
Buck, H. R. 1 bound vol., 8 pam.
Central London Ry. Co. 1 pam.
Colorado-Agri. Exper. Station. 2 pam.
Connecticut-R. R. Commrs. 1 bound vol.
Connecticut-Shell-fish Comm. 1 pam.
Cornell Univ. 1 vol.
Cuba-Secretaria de Obras Publicas. 1 vol.
Delft Technische Hoogeschool. 2 pam.
Deutsch-Amer. Tech. Verband. 2 pam.
Fall River, Mass.-City Engr. 6 pam.
Ferraz, L. C. 1 pam.
Germany-Kaiserliche General Direktion der Eisenbahnen in Elsass-Lothringen. 1 vol.
Great Eastern Ry. Co. 1 pam.
Great Northern & City Ry. Co. 1 pam.
Harvard Univ. 1 bound vol.
Hodgdon, Frank W. 1 bound vol.
Herschel. Clemens. 2 pam.
Illinois-State Water Survey. 1 pam.
Institution of Civ. Engrs. 1 bound vol., 3 pam.
Institution of Engrs. and Shipbuilders of Scotland. 1 bound vol.
Inter. Assoc. of Mun. Electricians. 1 bound vol., 1 pam.
Japan-Imperial Govt. Rys. 2 pam., 1 map.
Johnson, G. A. 1 pam.
Lake Mohonk Conference of Friends of the Indian and other Dependent Peoples. 1 pam.
Liverpool Eng. Soc. 1 bound vol.
Liverpool Overhead Ry. Co. 1 pam.
London, Brighton & South Coast Ry. Co. 1 pam.
London, Tilbury & Southend Ry. Co. 1 pam.
Madras, India-Public Works Dept. 1 pam.
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BY PURCHASE

Report with Regard to Civic Affairs in the City of Cedar Rapids, Iowa, with Recommendations for City Improvement and Beautification. By C. M. Robinson. The Torch Press. Cedar Rapids, 1908.

Repertorium der Technischen Journal=literatur. Herausgegeben im Kaiserlichen Patentamt, Jahrgang 1907. Carl Heymanns. Berlin, 1908.

SUMMARY OF ACCESSIONS

From January 13th to February 9th, 1909.

Donations (including 7 duplicates)	143
By purchase	2
Total	145

MEMBERSHIP

ADDITIONS

(January 13th to February 9th, 1909.)

MEMBERS

		Date of Membership.
ALDEN, HERBERT CLARENDON. Asst. Engr., Bureau of Sewers, Jerome Ave. and Macombs Rd., Borough of the Bronx, New York City	Assoc. M. M.	Mar. 1, 1893 Feb. 2, 1909
BULL, GEORGE MAIRS. Hydr. Engr., Denver Reservoir Irrig. Co., 705 Ideal Bldg., Denver, Colo		Feb. 2, 1909
HARRINGTON, FERDINAND FINNEY. Bridge Engr., The Virginian Ry. Co., National Bank of Commerce Bldg., Norfolk. Va	Assoc. M. M.	May 3, 1899 Feb. 2, 1909
HAUGH, JAMES CHARLES. Res. Engr., New Orleans & North Eastern R. R., Press and Levee Sts., New Orleans, La		Feb. 2, 1909
HORNSBY, ROBERT WRIGHT. Casilla 162, Valparaiso, Chili		Nov. 4, 1908
KNIGHT, RICHARD WARREN. Contr. Engr., McClintic-Marshall Constr. Co., Park Bldg., Pittsburg, Pa		Feb. 2, 1909
MANAHAN, ELMER GOVE. With Hering & Fuller, 170 Broadway, New York City	Assoc. M. M.	Oct. 7, 1903 Feb. 2, 1909
MARR, WILLIAM WALTER. 1893 Gladys Ave., Chicago, Ill		Feb. 2, 1909
MAXIMOFF, SERGIUS PAVLOVITCH. Engr. to the Imperial Russian Govt.; Asst. at the Imperial Inst. of Public Ways at St. Petersburg, Zabalkanski pr. 21, St. Petersburg, Russia	Assoc. M. M.	April 1, 1903 Sept. 1, 1908
MOORE, JOHN WILLIAM. 831 Morgan St., Rushville, Ind		Feb. 2, 1909
NEWTON, RALPH EELLS. Pres., Newton Engineering Co., 434 Jackson St., Milwaukee, Wis	Assoc. M. M.	Jan. 8, 1902 Feb. 2, 1909
WALMSLEY, WALTER NEWBOLD. Gen. Mgr., São Paulo Tramway, Light & Power Co., Ltd., São Paulo, Brazil		Dec. 1, 1908

ASSOCIATE MEMBERS

BLAMPHIN, ARTHUR MERRICK NEWBERRY. Asst. City Engr., Room 19, City Hall, New Orleans, La	Jun. Assoc. M.	Dec. 3, 1907 Feb. 2, 1909
BROWN, ALFRED THOMAS. Dept. of Highways, Borough of the Bronx, Cor. 177th St. and 3d Ave., New York City	Jun. Assoc. M.	Nov. 1, 1904 Jan. 5, 1909
EDWARDS, CHARLES MILTON. First Asst. Engr.. Dept. of N. Y. State Engr., 43 Triangle Bldg., Rochester, N. Y.		Sept. 2, 1908
FOWLER, ROBERT LAMBERT. 155 State St., Perth Amboy, N. J		Feb. 2, 1909

ASSOCIATE MEMBERS (*Continued*).

		Date of Membership.
GODDARD, HERBERT WILLARD. Engr. and Contr. (Cessna & Goddard), 178 Devonshire St., Boston, Mass.	Jun. Assoc. M.	Jan. 3, 1905 Jan. 5, 1909
LAMBERT, BYRON JAMES. Prof. of Structural Eng., State Univ. of Iowa, Iowa City, Iowa		Jan. 5, 1909
LEEFE, FREDERICK EWBANK. Care, U. S. Engr. Office, Baltimore, Md.		May 6, 1908
MACNAUGHTON, ERNEST BOYD. Care, MacNaughton, Raymond & Lawrence, Archts. and Engrs., Portland, Ore.		Oct. 7, 1908
PAIGE, JASON. Am. Bridge Co. of N. Y., 1308 Commercial National Bank Bldg., Chicago, Ill.	Jun. Assoc. M.	Feb. 5, 1907 Feb. 2, 1909
PIERCE, GEORGE ABEL. Asst. Engr., Champaign County, 120 East Church St., Urbana, Ohio		Jan. 5, 1909
SANGER, EDMUND PHIPPS. 301 S. 4th Ave., Mt. Vernon, N. Y.		Feb. 2, 1909
SMITH, ALBERT ORANGE. Civ. and Landscape Engr., Port Jefferson, N. Y.	Jun. Assoc. M.	May 2, 1905 Feb. 2, 1909
TALLMAN, LEROY. Portsmouth, R. I.		Feb. 2, 1909
TAYLOR, GRANVILLE LEWIS. Engr., Pittsburg Plant, McClintic-Marshall Const. Co.; Res., 119 Trenton Ave., Wilkinsburg, Pa.		Feb. 2, 1909
TÖNNESEN, FRIDTHJOV LAURITZ MARTIN. Hallingbury, Camp P. O., Jamaica		June 3, 1908

ASSOCIATE

BROMLEY, ALBERT HENRY, JR. Concrete Engr., 5113 Market St., Philadelphia, Pa.	Feb. 2, 1909

JUNIORS

BRENNAN, JOSEPH LAWRENCE. Brown Station, N. Y.	Jan. 5, 1909
DUBOIS, GUSTAVO ADOLFO. Constitucion 22, Matanzas, Cuba	Sept. 1, 1908
GREEN, CLARENCE JASPER. 907 Monroe St., Ann Arbor, Mich.	Sept. 1, 1908
GRINDROD, IRVIN SUTTON. 33d and Clearfield Sts., Philadelphia, Pa.	Feb. 2, 1909
HATHAWAY, CLIFFORD MURRAY. Draftsman, R. I. State Board of Public Roads, 50 Park St., Providence, R. I.	Sept. 1, 1908
PRITCHARD, JOHN CHARLES. Deputy State Highway Engr. of Mo., Columbia, Mo.	Sept. 1, 1908
RUSSELL, ALEXANDER STUART. Asst. Engr., Bureau of Public Works, Manila, Philippine Islands	Oct. 6, 1908
TILLIT, PEDRO ERNESTO. P. O. Box 1207, Lima, Peru	Oct. 6, 1908
WILLIAMS, HAROLD S. Caldwell, Idaho	Sept. 1, 1908
WILMOT, JAMES. Office of Public Roads, Washington, D. C.	Sept. 1, 1908

RESIGNATIONS

MEMBER	Date of Resignation.
MICHELL, THEOPHILUS	December 31, 1908

ASSOCIATE MEMBER	
BURNS, JAMES FERGUSON	February 2, 1909

JUNIORS	
BRASLOW, BARNETT	February 2, 1909
O'NEAL, ALFRED MOORE, JR	February 2, 1909

DEATHS

ADGATE, GEORGE. Elected Member, April 1st, 1896; died January 25th, 1909.

AMBROSE, WILLIAM CREELMAN. Elected Member, April 4th, 1888; died January 3d, 1909.

BOYLE, OLIN MCCLINTOCK, JR. Elected Junior, March 5th, 1907; died August 19th, 1908.

CRAIGHILL, WILLIAM PRICE (*Past-President*). Elected Member, October 7th, 1885; Honorary Member, March 23d, 1896; died January 18th, 1909.

GRAHAM, JOSEPH MARSHALL. Elected Member, April 4th, 1900; died February 4th, 1909.

QUINETTE DE ROCHEMONT, *Baron* EMILE THEODORE. Elected Member, January 3d, 1894; died December 8th, 1908.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(January 12th, to February 8th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $4.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian,* London, England.
(58) *Proceedings,* Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions,* Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 20c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England.
(70) *Engineering Review,* New York City, 10c.
(71) *Journal,* Iron and Steel Inst., London, England.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
(77) *Journal,* Inst. Elec. Engrs., London, England.
(78) *Beton und Eisen,* Vienna, Austria.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal,* Berlin, Germany.
(83) *Progressive Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Roadmaster and Foreman,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.

LIST OF ARTICLES

Bridge.

The Curzon Bridge at Allahabad.* Robert Richard Gales, M. Inst. C. E. (63) Vol. 174.
The Netravati Bridge at Mangalore.* Archibald Scott Napier, M. Inst. C. E. (63) Vol. 174.
The King Edward VII Bridge, Newcastle-on-Tyne.* Frank William Davis and Cyril Reginald Sutton Kirkpatrick, Assoc. MM. Inst. C. E. (63) Vol. 174.
The New Bridge Crossing the Mississippi River at Clinton, Ia.; Chicago & North-western Ry.* F. H. Bainbridge. (13) Jan. 21.
Rio Conchos Bridge of the Kansas City, Mexico & Orient.* W. W. Colpitts, M. Am. Soc. C. E. (15) Jan. 22.
The Rocky River Concrete Bridge near Cleveland, O.* (14) Jan. 23.
Cost of Concrete Bridges. Henry H. Quimby. (Paper read before the National Assoc. of Cement Users.) (14) Jan. 23.
Cost of Small Concrete Piers.* (14) Jan. 23.
The Footwalks of the Blackwell's Island Bridge.* F. W. Abbott, M. Am. Soc. C. E., and J. P. H. Perry. (13) Jan. 28.
Live Loads and Working Stresses in Railway Bridges.* Conrad Gribble, A. M. I. C. E. (10) Feb.
A Comparison of Bridge Waterways.* Daniel B. Luten. (Paper read before the Ind. Eng. Soc.) (60) Feb.
Methods and Cost of Constructing a Plate Girder Bridge with Concrete Piers.* (86) Feb. 3.
An Ornamental Bascule Bridge; Trunnion Bascule across Coopers Creek, at Federal St., Camden, N. J.* (13) Feb. 4.
Conventional Live-Load Positions for Maximum Stresses in Arches. Wm. Cain, M. Am. Soc. C. E. (13) Feb. 4.
Central of New Jersey Bridge Renewal at Bethlehem.* (15) Feb. 5.
Les Ponts Modernes.* José-Eugenio Ribera. (From *Revista de Obras Publicas.*) (43) May, 1908.
Le Pont Neuf, Pont-Levant à Manœuvre Electrique au Port de Cette.* Herrmann. (43) May, 1908.

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*Illustrated.

Bridge—(Continued).

Pont sur la Maumee (Ohio).* (84) Dec.
Ponts en Poutrelles Enrobées.* Siegler. (38) Jan.
Pont en Béton Armé sur le Rhône, à Pyrimont (Ain et Haute-Savoie).* G. Espitallier. (33) Jan. 16.
Strassenbrücke über die Memel bei Tilsit, Königin-Luisen-Brücke.* Dieckmann. (49) Vol. 1-3, 1909.
Eine neue Graphische Berechnungsmethode für Beiderseits Eingespannte Träger mit Veränderlichem Trägheitsmoment. Rich. Wuczkowski. (78) Jan. 8.

Electrical.

Report of Committee on the Magnetic Testing of Iron and Steel. (Amer. Soc. for Testing Materials.)* (89) Vol. 8.
Uniformity in Magnetic Testing and in the Specification of Magnetic Properties.* Chas. W. Burrows. (89) Vol. 8.
The Development of the Electric Piano Player.* John F. Kelly. (3) Jan.
The Telegraphone. Charles K. Fankhauser. (3) Jan.
The Electrical Qualities of Porcelain, with Special Reference to Dielectric Losses.* H. F. Haworth. (Abstract of paper read before the Royal Soc.) (73) Jan. 1.
The Hemsjö Power Company, Sweden.* P. Frenell. (73) Serial beginning Jan. 8.
The Post Office Radio-Telegraph Station at Bolt Head.* (27) Jan. 8.
System of the Rochester Railway & Lighting Company.* (27) Jan. 14.
Factors Governing the Space Utilization of Electro-magnetic Windings.* Charles R. Underhill. (27) Jan. 14.
A Directive System of Wireless Telegraphy. E. Bellini and A. Tosi. (Abstract of paper read before the Physical Soc.) (73) Jan. 15.
Deformation of Pressure Curves due to Load in Alternating Current Generators. Egon Siedek. (Abstract from *Elek. u. Maschinenbau.*) (73) Jan. 15.
Insulation of High-Tension Transmission Lines.* F. S. Denneen. (Abstract of paper read before the Central Elec. Club.) (73) Jan. 15.
Station No. 3 of the Rochester Railway & Light Company.* (27) Jan. 21.
The Influence of Frequency on the Equivalent Circuits of Alternating-Current Transmission Lines.* A. E. Kennelly. (27) Jan. 21
Representation of Synchronous Converter Phenomena.* Olin J. Ferguson. (27) Jan. 21.
The Improvement of Power Factor in Alternating-Current Systems.* Miles Walker. (Abstract of paper read before the Inst. Elec. Engrs.) (73) Jan. 22.
Korn's Apparatus for Photographic Transmission.* Louis Dubois. (73) Jan. 22.
Electricity and its Application to the Reduction of Waste.* A. B. Gridley. (Abstract of paper read before the Cleveland and Durham Eng. Soc.) (73) Jan. 22.
Metallic Filament Lamps.* W. A. Barnes. (Abstract of paper read before the Manchester Assoc. of Engrs.) (47) Serial beginning Jan. 22.
The Power System of the Rochester Railway & Light Company.* (17) Jan. 23; (27) Jan. 28.
Fluctuations in the Speed of the Rotors of Alternators Operating in Parallel.* George H. Shepard. (27) Jan. 28.
A New System of Wireless Telephony.* A. Carletti. (73) Jan. 29.
Parallel Operation of Alternators.* E. Rosenberg. (Abstract of paper read before the Inst. Elec. Engrs.) (73) Serial beginning Jan. 29.
Sending Pictures by Wireless.* Alfred Gradenwitz. (19) Jan. 30.
Prime Movers.* Charles P. Steinmetz. (42) Feb.
Electric Heating. W. S. Hadaway, Jr. (42) Feb.
Length and Voltage of Transmission Lines.* Alton D. Adams. (10) Feb.
Electric Motors in a Brewery.* William Keily. (27) Feb. 4.
Comparative Tests of Transformers.* A. C. Scott. (27) Feb. 4.
Cost of Energy in Nine Central Stations Operating Mixed Equipment. (27) Feb. 4.
Note sur le Télé-Stéréographe.* E. Belin. (92) Dec.
Grue Electrique Pivotante Fixe de 30-10 Tonnes du Port de Nice.* Ch. Dantin. (33) Jan. 9.
Die Gleichstrom-Querfeldmaschine und Ihre Anwendungen, Insbesondere für Elektrische Zugbeleuchtung.* Richard Neumann. (48) Jan. 23.

Marine.

The Two-Tone Vibrating Transmitter and Inductive Signalling. Edward Raymond-Barker. (73) Jan. 15.
The Efficiency of Marine Engines and Propellers.* J. Hamilton Gibson. (12) Jan. 29.
Magnetic Survey Yacht *Carnegie*.* (95) Feb.
Piping up a Merchant Vessel for Steam Heat.* Allan Dale. (95) Feb.

*Illustrated.

Marine—(Continued).

The Development and Present Status of the Experimental Model-Towing Basin.* H. A. Everett. (95) Feb.
A New Reversing Motor for Launches and Yachts.* G. O. M. Olsson. (Comp. from *Teknisk Tidskrift.*) (95) Feb.
Application du Moteur Diesel à la Navigation.* A. Bochet. (32) Nov.
La Traction Electrique des Bateaux sur les Canaux.* P. du Bousquet. (32) Nov.
Der Doppelschrauben-Personen- und Frachtdampfer *Prinz Friedrich Wilhelm.* (48) Serial beginning Jan. 2.

Mechanical.

The Temperature Gradient in De Laval Steam-Nozzles.* Cyril Batho. (63) Vol. 174.
Method of Obtaining a Truly Circular and Uniform Chill in Rolls.* Thomas D. West. (89) Vol. 8.
The Influence of Fine Grinding on the Physical Properties of Portland Cement. Richard K. Meade. (89) Vol. 8.
Testing Lubricating Oils. Henry Souther. (89) Vol. 8.
Special Features of a Recently Installed 600 000-Lb. Universal Testing Machine.* Thorsten Y. Olsen. (89) Vol. 8.
New Forms of Pendulum Testing Machines.* Thorsten Y. Olsen. (89) Vol. 8.
Commercial Results in the Purchase of Coal on Specifications.* J. E. Woodwell. (89) Vol. 8.
The Use of the Extensometer in Commercial Work.* T. D. Lynch. (89) Vol. 8.
The Direct Production of Copper Tubes, Sheets and Wire.* Sherard O. Cowper-Coles. (75) July.
The Evolution and Methods of Manufacture of Spur-Gearing.* Thomas Humpage. (75) July.
Inclined Retort Coal- and Coke-Handling Plant at Bristol. William Stagg. (75) July.
The Destructive Agencies Acting upon Refractories. A. V. Bleininger. (76) Jan.
Salt-Glazed Blue Brick. D. Cobb, A. M. I. M. E. (76) Jan.
Modern Practice in Making Dry Press-Brick. John J. Moroney. (76) Jan.
An "Up-to-the-Minute" Yard, Scott System and Gas-Fired Kiln at Spinks Plant, Covington, Kentucky.* (76) Jan.
Gasification of Coal in the Producer.* William Young. (66) Serial beginning Jan. 5.
The Collapse of Tubes under External Pressure.* S. E. Slocum. (11) Jan. 8.
Temperley Transporters.* (11) Serial beginning Jan. 8; (57) Jan. 15.
Rock Drill Making and Sharpening by Machine and Treatment of Drill Steel.* W. E. Kimber. (Paper read before the Transvaal Inst. of Mech. Engrs.) (22) Jan. 8.
The Coal-Screening and Conveying Plant at the Collieries of Messrs. Burn-Yeat Brown and Co., Nine Mile Point, near Newport, Monmouth.* (57) Jan. 8.
New Power Plant of Carnegie Institute.* Thomas Wilson. (64) Jan. 12.
A. B. C. of Combustion in Boiler-Settings. Jacques Abady, M. Inst. M. E. (Paper read before the Soc. Mech. and Elec. Engrs.) (66) Jan. 5.
London Sliding Scale for Gas. Wm. D. Marks. (83) Jan. 15.
The Commercial Truck vs. the Horse.* Benjamin Rogers. (46) Jan. 16.
High Tension Ignition by Magnets (Automobiles).* Roger B. Whitman. (46) Jan. 16.
Hampton Power Plant of the D., L. & W. R. R.* Warren O. Rogers. (64) Jan. 19.
Ferro-Concrete Coal Storage Plant.* (57) Jan. 22.
A Modern Coke-Oven Gas Plant: the Bargoed Colliery Installation of the Powell Duffryn Steam Coal Co.* (27) Jan. 22.
The Laws of Heat Transmission in Steam Boilers, as deduced from Experiment.* John T. Nicolson. (Paper read before the Jun. Inst. Engrs.) (47) Serial beginning Jan. 22.
A Heavy Oil Motor Wagon.* (12) Jan. 22.
The Gröndal-Kjellin and Röchling-Rodenhauser Electric Steel Furnaces.* (11) Jan. 22.
5,000-Ton Press for Dynamic Compression of Fluid Steel.* (11) Jan. 22.
Proportions of Diameter and Stroke in Petrol-Engines.* (11) Jan. 22.
Coal Briquetting, with Special Reference to the Briquetting Machine Built and Used by the Standard Fuel Company, Birmingham, Ala.* R. M. Hale. (20) Jan. 28.
A Comparison of American High-Speed Runners for Water Turbines.* S. J. Zowski. (13) Jan. 28.
Group of Reinforced-Concrete Buildings for a Wood Distillation Plant, Donald, Ont.* (13) Jan. 28.
Coal Shipping Arrangements at St. John's Collieries, Normanton.* (22) Jan. 29.

*Illustrated.

Mechanical—(Continued).

The Utilisation of Exhaust Steam in Low-Pressure Turbines.* R. P. Halliwell. (Paper read before the Rugby Eng. Soc.) (**47**) Serial beginning Jan. 29.
Trials of Two Pairs of Compound Engines.* Michael Longridge, M. Inst. C. E. (From Report of Chf. Engr., the Brit. Engine, Boiler and Elec. Insurance Co., Ltd.) (**47**) Serial beginning Jan. 29.
Experiments with Rolling Mill Plant.* F. Carleton Anderson. (Abstract of paper read before the Manchester Assoc. of Engrs.) (**47**) Serial beginning Jan. 29.
Form and Stability of Aeroplanes; researches Supplementing those of Lilienthal and Langley.* W. R. Turnbull. (From *Physical Review.*) (**19**) Jan. 30.
The Manufacture of High-Speed Steel.* O. M. Becker. (**10**) Feb.
Prime Movers.* Charles P. Steinmetz. (**42**) Feb.
Lay Out of a Horizontal Return Tubular Boiler 18 Feet Long by 72 Inches Diameter.* R. R. Sturgeon. (**94**) Feb.
Reinforced Openings in Boiler Shells.* S. F. Jeter. (**94**) Feb.
Energy in a Pound of Steam.* Fred R. Low. (**64**) Feb. 2.
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PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STEEL SHEETING AND SHEET-PILING

By L. R. Gifford, Assoc. M. Am. Soc. C. E.

To be Presented March 17th, 1909.

The subject of steel sheeting and sheet-piling, for some time past, has been receiving a great deal of attention from both engineers and contractors, and the many advantages of these materials, in comparison with the types of timber construction customarily used, is readily appreciated.

The general idea of metal piles is not new, but only within the last five or six years has any considerable progress been made in developing a form which was commercially practical and at the same time sufficiently economical to permit of its use except in special cases.

The earlier forms were of cast-iron, and a very interesting paper, by Mr. Michael A. Borthwick, published in 1836 and reprinted in "Ordinary Foundations," by Charles Evan Fowler, M. Am. Soc. C. E., gives drawings of a number of these forms, and describes their use at considerable length.

That cast-iron sheet-piling did not continue to be used to any great extent was doubtless due to its greater cost as compared with timber sheet-piling, as well as its general unsuitableness for work of this

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

character. It is rather remarkable, however, when it is considered that the difficulty of securing a satisfactory sheet-piling is one of the very troublesome problems with which the construction engineer has to deal, that other forms of metal sheet-piling were not devised, particularly in view of the apparent success which attended the use of the cast-iron piling. When cast-iron piling was first used, however, there was a much greater difference between the cost of metal and timber piling than at present, particularly in the United States, where timber was abundant.

At the present time there are a number of forms of steel piling on the market, and the many advantages which they possess have given them a deserved popularity, which has undoubtedly been augmented by the rapidly increasing cost of timber.

The writer has recently had occasion to study steel "sheeting" and "sheet-piling" quite carefully, and, in presenting briefly the result of these studies to the Society, does so in the hope that the subject may prove of sufficient importance to bring out a further discussion.

The best definitions of the terms "sheeting" and "sheet-piling" are to be found in the paper* by J. C. Meem, M. Am. Soc. C. E., entitled "The Bracing of Trenches and Tunnels, with Practical Formulas for Earth Pressures." Mr. Meem defines "sheeting" as that class of sheathing which is set in or driven coincidentally with the excavation; and "sheet-piling" as that class of sheathing which is driven ahead of the excavation, or beyond its final limits.

In what follows, an attempt will be made to determine the essentials of a satisfactory steel piling, and an analysis of some of the principal sections in use at the present time will be made.

The construction engineer's requirements for sheeting and sheet-piling are about as follows:

First. Strength, to resist earth or water pressure after the piling is in place, without an excessive amount of bracing;

Second. Stiffness, to enable it to be driven without buckling or springing under the blows of the driving hammer;

Third. Water-tightness, to prevent seepage.

On the other hand, the manufacturer will require a section which can be manufactured practically and economically, in order that the first cost shall not be excessive.

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* *Transactions*, Am. Soc. C. E., Vol. LX, p. 1.

The various forms of steel sheet-piling which are most used at the present time can be placed in three groups, as shown by the sections on Fig. 1. For convenient comparison, these sections are drawn to the same scale.

The first of the assumed requirements—that the piling must have lateral strength to support earth or water pressure—will now be considered. It will be seen that from this point of view the problem stated generally is to support a continuous and uniformly varying load over a flat surface, and the most convenient measure of the strength is the "section modulus," on the basis of 1 ft. width of the piling in place. This method involves the assumption that the sections when interlocked act as a unit. The diagram, Fig. 2, shows graphically how certain of the sections shown on Fig. 1 compare in this respect.

In constructing the diagram the vertical ordinates are the weight per square foot of the piling assembled, and the horizontal ordinates are the section moduli, expressed in cubic inches for a section 1 ft. in width. An inspection of the diagram will indicate the relative economy of the sections as regards the ratio of weight to strength.

The Lackawanna type would doubtless show approximately the same section modulus as the United States type, as the over-all dimensions of the flanges are about the same. The spring-lock type, as will be noted from an inspection of the drawings, depends on the material in the joints for its lateral strength, but, as the plate forming the body of the pile may be stiffened by standard rolled sections riveted thereto, it is difficult to include this type in the comparison.

The second requirement—that of stiffness to resist buckling while being driven—can be measured by the least radius of gyration of a single piece to be driven, and this is shown graphically on the diagram, Fig. 3. In this diagram the vertical ordinates, as in the previous diagram, represent the weight per square foot, while the horizontal ordinates are the least radii of gyration, in inches.

The third requirement—that of water-tightness—is met most economically by those sections which do not require auxiliary packing pieces, such as wood fillers at the joints, or caulking strips, to render them tight.

From the manufacturers' standpoint, the sections illustrated under Group 1 are desirable for the reason that they are composed of commercial structural shapes, which feature, however, is to a certain extent

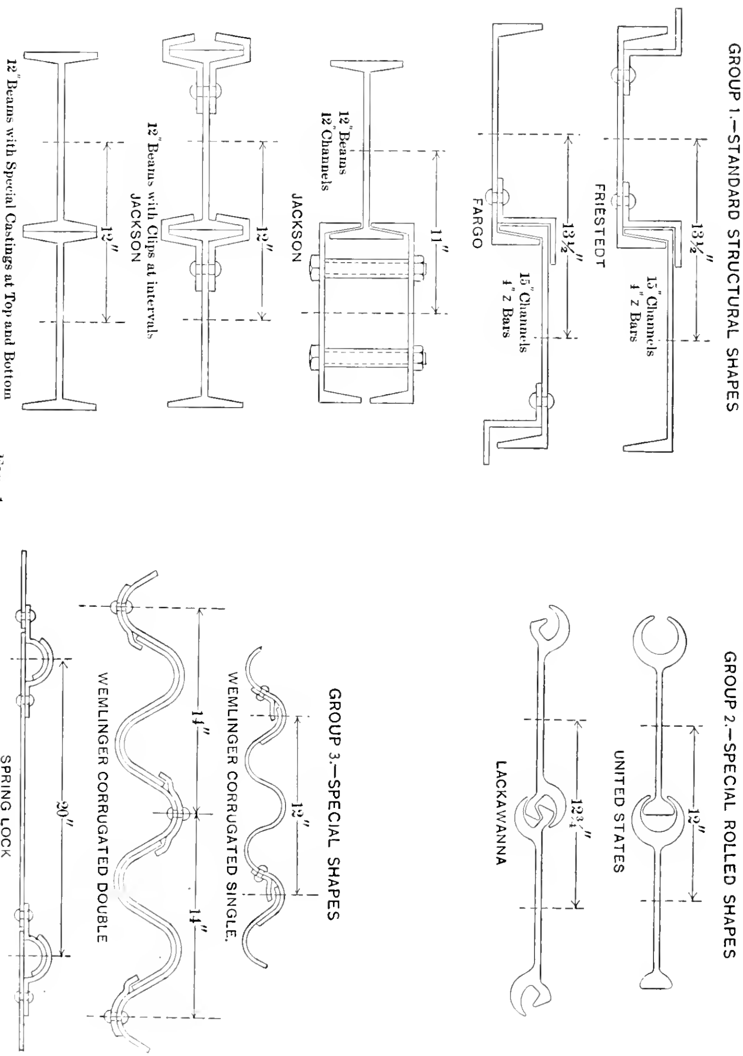

GROUP 1.—STANDARD STRUCTURAL SHAPES
13½″
15″ Channels
4″ Z Bars
FRIESTEDT
13½″
15″ Channels
4″ Z Bars
FARGO
11″
12″ Beams
12″ Channels
JACKSON
12″
12″ Beams with Clips at intervals
JACKSON
12″
12″ Beams with Special Castings at Top and Bottom
NYE
GROUP 2.—SPECIAL ROLLED SHAPES
12″
UNITED STATES
12¾″
LACKAWANNA
GROUP 3.—SPECIAL SHAPES
12″
WEMLINGER CORRUGATED SINGLE.
14″
14″
WEMLINGER CORRUGATED DOUBLE
20″
SPRING LOCK

FIG. 1.

offset by the fact that considerable shop work is necessary in order to rivet the various pieces together.

Special rolled shapes, Group 2, were designed in order to eliminate all shop work and permit of the sections being used as they come from the rolling mill, particular attention being paid to the interlocking feature.

The sections classified under Group 3 require considerable shop work, and are undoubtedly the more expensive of the various types on the basis of cost per pound of material, but this is offset by the fact that a much lighter and stronger section can be secured, which results, generally, in a lower cost per square foot of material in place, as compared with other types.

At the present time no satisfactory method is in general use for specifying the requirements of steel sheeting and sheet-piling. One method is to specify a minimum thickness; another is to specify a minimum weight per square foot after assembling. Neither of these methods is satisfactory, as both entirely overlook what, in the writer's opinion, are the prime essentials, viz., strength, stiffness, and water-tightness.

It is evident that the specification of a minimum thickness can be met by using a flat plate, but this, of course, would be entirely unsuitable, unless some means of properly stiffening the plate is provided, in order to enable it to be driven into place, as well as a proper method of joining the plates at the edges. The requirement of a minimum weight per foot can be met by increasing the weight at the joint and reducing the thickness of the metal between the joints, which is equally undesirable.

The writer considers that a proper and reasonable specification for steel sheeting and sheet-piling is essential and should be used by engineers whenever this material is to be used.

The problem of computing the strength required in sheeting and sheet-piling is a very difficult and complicated one, as is well illustrated by Mr. Meem's paper, previously referred to, and the discussion in connection therewith. It is very necessary, however, that any specifications should take this very important feature into consideration, and the following method, it is believed, will provide a satisfactory basis for specifying the minimum strength required.

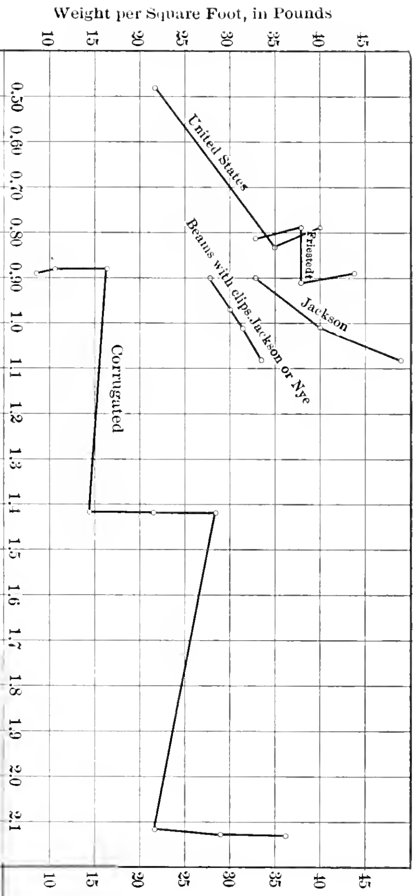

Weight per Square Foot, in Pounds
Least Radius of Gyration, in inches
0.50 0.60 0.70 0.80 0.90 1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2.0 2.1
10 15 20 25 30 35 40 45
United States
Friestedt
Jackson
Beams with clips, Jackson or Nye
Corrugated

FIG. 3

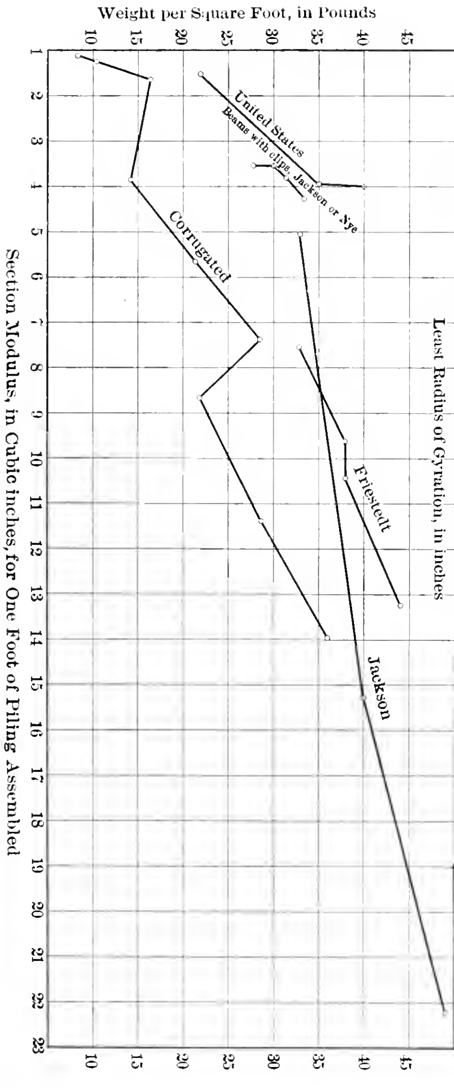

Weight per Square Foot, in Pounds
Section Modulus, in Cubic inches, for One Foot of Piling Assembled
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23
10 15 20 25 30 35 40 45
United States
Beams with clips, Jackson or Nye
Corrugated
Friestedt
Jackson

FIG. 2

The assumptions made are:

First. The pressure to be supported is hydrostatic;

Second. The sheeting or sheet-piling is in the condition of a simply supported beam, one support being the bracing at the top, and the other the ground at the bottom;

Third. The load is uniformly increasing from zero at the top to a maximum at the bottom.

Fourth. For permanent work, the allowable fiber stress should not exceed 16 000 lb. per sq. in., which provides a safety factor of 4; for temporary work, however, a fiber stress of 20 000 would not be excessive, and this provides for a safety factor of approximately 3.

Expressing the foregoing assumptions in a formula, we have:

$$S = \frac{4.01\ d^3}{16\ 000} \text{ for permanent work;*}$$

$$S' = \frac{4.01\ d^3}{20\ 000} \text{ for temporary construction;*}$$

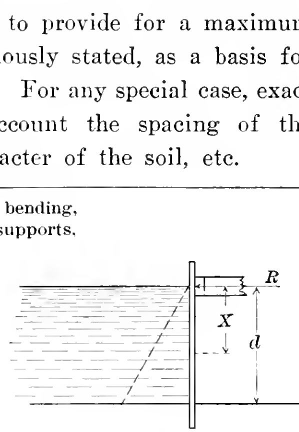
when S and S' = section moduli in inches^3;

d = depth of water supported, in feet.

Proper bracing, of course, is essential, and the values obtained from these formulas are intended merely to provide for a maximum theoretical condition, to be used, as previously stated, as a basis for specifying the minimum strength required. For any special case, exact figures should be made, taking into account the spacing of the rangers and bracing to be used, the character of the soil, etc.

---

*X = distance from support to point of maximum bending,
d = span of beam, *i.e.*, length of piling between supports,
R = reaction,
M = bending moment at distance X,
S = section modulus of beam,
f = allowable fiber stress,
W = total load.



$$W = \frac{62.5\ d^2}{2}$$

$$R = \frac{62.5\ d^2}{6}$$

$$X = \frac{d\sqrt{3}}{3}$$ See "Treatise on the Resistance of Materials" Wood. p. 195.

$$M = \frac{62.5\ d^2}{6} \times \frac{d\sqrt{3}}{3} - \frac{62.5\ d^2}{2} \times \left(\frac{d\sqrt{3}}{3}\right) \times \frac{1}{3d^2}$$

$$= 4.01\ d^3$$

$$M = f\,S = 4.01\ d^3$$

$$S = \frac{4.01\ d^3}{f}.$$

With this explanation of the paragraph relating to the strength, the writer offers the following specifications, which, it is believed, will provide steel sheeting or sheet-piling, without designating any particular form, which will meet any reasonable requirements.

Specifications.

Material.—All steel used shall be in accordance with the Specifications of the American Association of Steel Manufacturers for Structural Steel for Buildings, Revised to February 6th, 1903.

Unit Strains.—The allowable unit strains per square inch in bending on extreme fiber shall not exceed the following values:

For temporary work, 20 000 lb.
For permanent work, 16 000 lb.

Strength.—The section used shall have a section modulus per foot, when assembled in place, of not less than the following:

	Permanent.	Temporary.
Lengths of 10 ft. or less	0.25	0.20
" from 10 to 15 ft.	0.85	0.68
" " 15 " 20 "	2.00	1.61
" " 20 " 25 "	3.92	3.13
" " 25 " 30 "	6.77	5.42
" " 30 " 35 "	10.75	8.60

Stiffness.—Single pieces driven separately shall have a ratio of length in inches to least radius of gyration of not more than 250.

Water-Tightness.—The sections when assembled in place shall be water-tight at the joints, without the use of auxiliary packing or caulking pieces, unless such packing or caulking pieces are made part of and are assembled with the sections of the sheeting or sheet-piling before driving.

Driving.—Care shall be taken in driving to prevent battering of the heads of the piles, and, if necessary to prevent this, a tight fitting cap shall be used. Driving shall be done, preferably, by steam or compressed-air hammers.

Applications.

Steel sheet-piling of rolled sections was first used in the United States in 1902, when the Jackson type of beams and channels was utilized in the construction of a coffer-dam for the foundation of the Randolph Street Bridge, Chicago, Ill. Since that time the use of steel sheet-piling for coffer-dams of all kinds has become quite general.

The largest example is, without doubt, the coffer-dam now under construction for the United States Government at Buffalo, N. Y. This coffer-dam encloses an area 887 by 200 ft., in which is to be constructed a ship lock in Black Rock Harbor and Channel. Two lines of steel sheet-piling of the Lackawanna type are being used, driven 30 ft. apart and connected by cross-walls at intervals of 30 ft. The pockets thus formed are to be filled with clay, before the water is pumped out of the coffer-dam.

A very economical application of steel sheet-piling is its use in connection with dams or embankments, where it is frequently driven to form a core-wall designed to prevent seepage. The first suggestion of its use for this purpose which the writer has seen was that made by the late George S. Morison, Past-President, Am. Soc. C. E., in his paper entitled "The Panama Canal."* In this paper Mr. Morison proposed to use steel sheet-piling of rolled sections in connection with the "Bohio Dam."

Probably the first actual application of steel sheet-piling for a core-wall was made by the Hackensack Water Company, near Hillsdale, N. J., the Friestedt type being used. A recent example is a dam at Scotland, Conn., for the Uncas Power Company, of Norwich, Conn. In this case an attempt was made to drive 6 by 8-in. yellow-pine sheet-piling of the ordinary tongued and grooved type fitted with malleable cast-iron chisel points. The soil was of a dense gravelly character, or, more properly, a hardpan, which the timber piling failed to penetrate, the piles brooming or splitting to such an extent as to be utterly worthless. Steel sheet-piling of the Wemlinger corrugated type, formed of $\frac{3}{16}$-in. and $\frac{1}{4}$-in. plates, was substituted and satisfactorily driven, although in some cases more than 4 000 blows of a steam hammer with a ram weighing 3 000 lb. were necessary in order to secure a penetration of 18 ft., due to the very compact character of the soil.

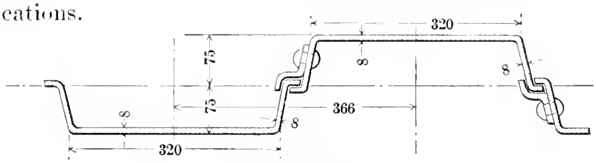
Another application of steel sheet-piling which is becoming more and more general is its use as shoring in the construction of building foundations, especially where the foundations must be carried below those of adjoining buildings or through soft or water-bearing soil. It is frequently possible by the use of steel sheet-piling to secure satisfactory foundations which would otherwise require the use of pneumatic caissons, which are very expensive.

The construction of mine shafts is another application in which

* *Transactions*, Am. Soc. C. E., Vol. L, p. 177.

steel sheet-piling has been used successfully. However, probably the largest field for the adoption of steel sheet-piling is in trench work, and, while it has been used to some extent for this purpose, it is only recently that a type of steel sheeting has been developed which is economical in comparison with wood sheeting. This is due to the fact that it is now possible to secure a steel sheeting weighing 10 lb. or even less per sq. ft. which can be driven in widths of 12 in., and when it is considered that this sheeting, with proper care, may be used practically indefinitely, the saving over the timber is evident.

Recently, in the construction of a sewer at Summit, N. J., 100 lin. ft. of 10-ft. Wemlinger corrugated sheeting was used on 8 000 ft. of sewer in a trench varying in depth from 9 to 22 ft., that is to say, the same sheeting was driven and pulled at least 80 times, and at the completion of the job was still in good condition. This sheeting was made of 16-gauge material, weighing 5 lb. per sq. ft. in place. Sheeting of the same type is now being used for sewer and conduit trenches in Washington, D. C., Buffalo and Rochester, N. Y., and a number of other locations.



LARSSEN PILING
Dimensions in Millimeters

FIG. 4.

A use to which the lighter forms of steel sheet-piling are especially adapted is in connection with levees. Driven down through the center of these it would effectually cut off any seepage under them and absolutely prevent their being undermined by crawfish, muskrats, or other boring animals. The cost of the steel sheet-piling used in this manner would be largely made up in the saving which could be effected by the elimination of the muck ditches and the greatly reduced section of the levee itself made possible thereby.

Still another application is in the construction of docks and bulkheads, and, while it has never been used for this purpose in the United States, as far as the writer is aware, it has been done in Germany, at Hohentor Harbor, near Bremen, and in the port of Bremen, where a considerable quantity of bulkhead work has been built of the Larssen type of rolled-steel piling, Fig. 4.

The Larssen type of steel sheet-piling was also used in the construction of the walls of two large ship-locks on the canal above Bremen, near Hamelingen.

In all these cases, the steel sheet-piling was driven along the bank, and longitudinal waling beams were afterward bolted to the piling, and the entire structure was secured near the top with anchor-rods fastened at one end through the waling beams and at the other to concrete slabs buried in the bank. The results are entirely satisfactory; the bulkheads present a very substantial appearance, and are much more economical in cost than concrete or masonry structures, while, at the same time, they are considerably more durable than timber bulkheads.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE SIXTH STREET VIADUCT, KANSAS CITY.

By E. E. Howard, Assoc. M. Am. Soc. C. E.

To be Presented April 7th, 1909.

The Sixth Street Viaduct, recently built, is a steel structure, about 8 000 ft. long, providing an elevated roadway and two car tracks across the broad, flat valley of the Kaw River to the high land on each side. It was designed to accommodate, and, by improved conditions for comfort, safety, and speed, to facilitate street-car and vehicular traffic; thus contributing to the unification of the social and commercial interests of Kansas City, Missouri, and Kansas City, Kansas.

The main viaduct and those portions of the side laterals or approaches now finished include, in general figures, 1 900 lin. ft. of roadway 38 ft. wide, 6 100 lin. ft. 30 ft. wide, 1 000 lin. ft. 24 ft. wide, and 1 000 lin. ft. 20 ft. wide, a total of 10 000 lin. ft. of roadway; also 8 500 lin. ft. of double-track railway. In this construction there have been used more than 120 000 lin. ft. of piles, 23 000 cu. yd. of mass concrete, 13 000 tons of steel, nearly 10 000 cu. yd. of reinforced concrete, 34 000 sq. yd. of asphalt pavement, besides minor and miscellaneous material. On one lateral, or side approach, where an incline could not be conveniently placed, there were installed two large electric traffic elevators of sufficient capacity to raise any vehicle and team.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Figs. 1 and 2, Plate II, are general views of the viaduct and the bridge.

Kansas City is located on the south side of the Missouri River, below one of its great bends, and lies in the States of Missouri and Kansas, with the crooked Kaw River flowing through it to join the greater stream. While commercially and socially possessed of common interests, the community has political and governmental divisions, due to its position on the border line of two States, and is divided into two distinct cities with their various suburban settlements.

The valley of the Kaw River, popularly called the "West Bottoms," is about a mile wide and is quite flat; it is skirted on each side by high bluffs and hills, while the channel of the river meanders through it, in its lower reaches lying close to the western bluff. The Missouri River has been gradually pushed northward, through the agency of dikes and mattresses, and has deposited, in the last fifteen years, a strip of land, about one-quarter of a mile wide, extending from side to side of the valley of its tributary. While practically the entire residence portions and the principal retail business districts of both cities are on the hills, there are to be found, crowded into the Kaw Valley, many commercial industries, packing houses, stock yards, manufacturing plants, wholesale warehouses, the freight stations of nearly all the thirty-four railroads entering the city, several large terminal freight yards, and the present union passenger station. As an essential adjunct, there is, throughout the bottoms, a complicated network of railway tracks, occupying streets and alleys as well as private rights of way.

The transaction of the business of the cities involves an enormous traffic into this valley from both sides, and all traffic from city to city must pass through it. A very large portion of the freight consigned for local consumption must be hauled out of the bottoms; some up-town distributing houses haul goods out, store them, and then haul them back; the packing houses, factories, and other industries are continually conveying their products to both cities; and, too, a great deal of the merchandise consumed in Kansas City, Kans., is hauled from Kansas City, Mo.

All this traffic is confronted by very inadequate passageways and very heavy grades from valley to hills, not less than 5% on the east side, and not less than 6% on the west side. Furthermore, on the east side there are from the northern part of the city only two possible

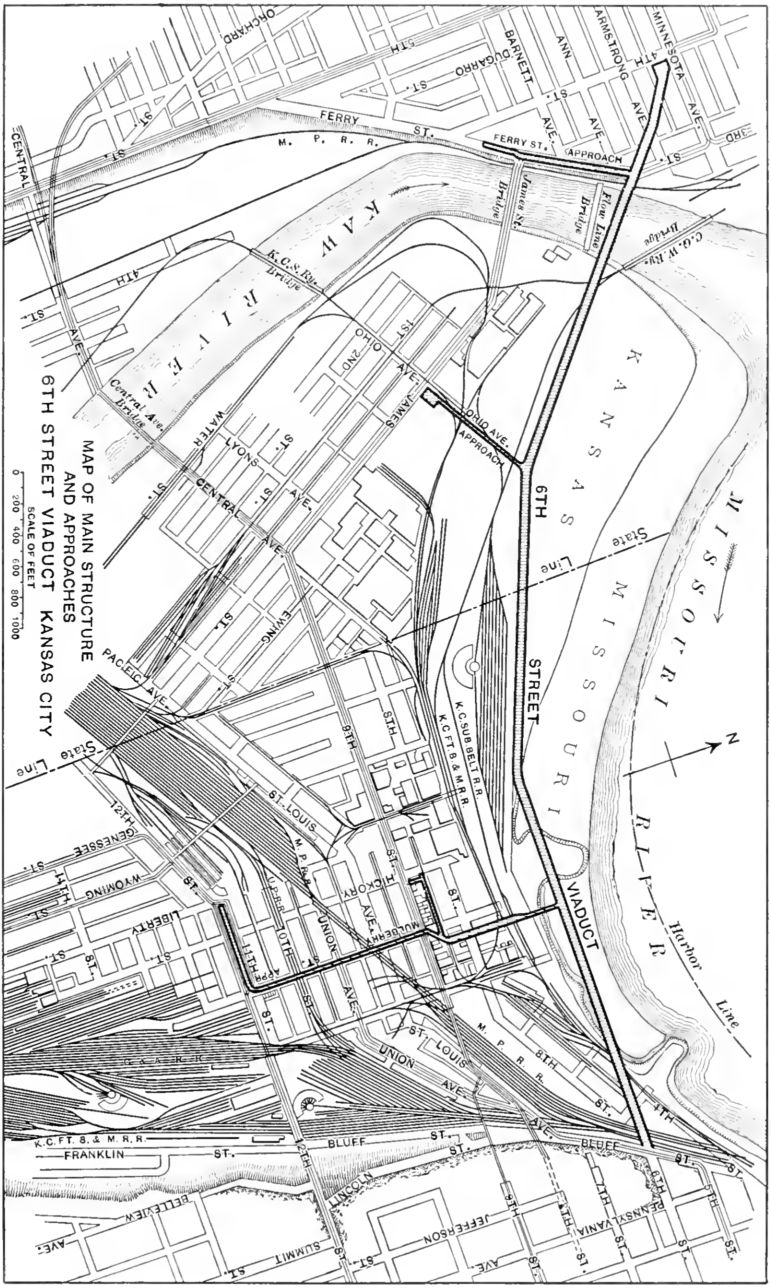

MAP OF MAIN STRUCTURE
AND APPROACHES
6TH STREET VIADUCT KANSAS CITY
0 200 400 600 800 1000
SCALE OF FEET

FIG. 1.

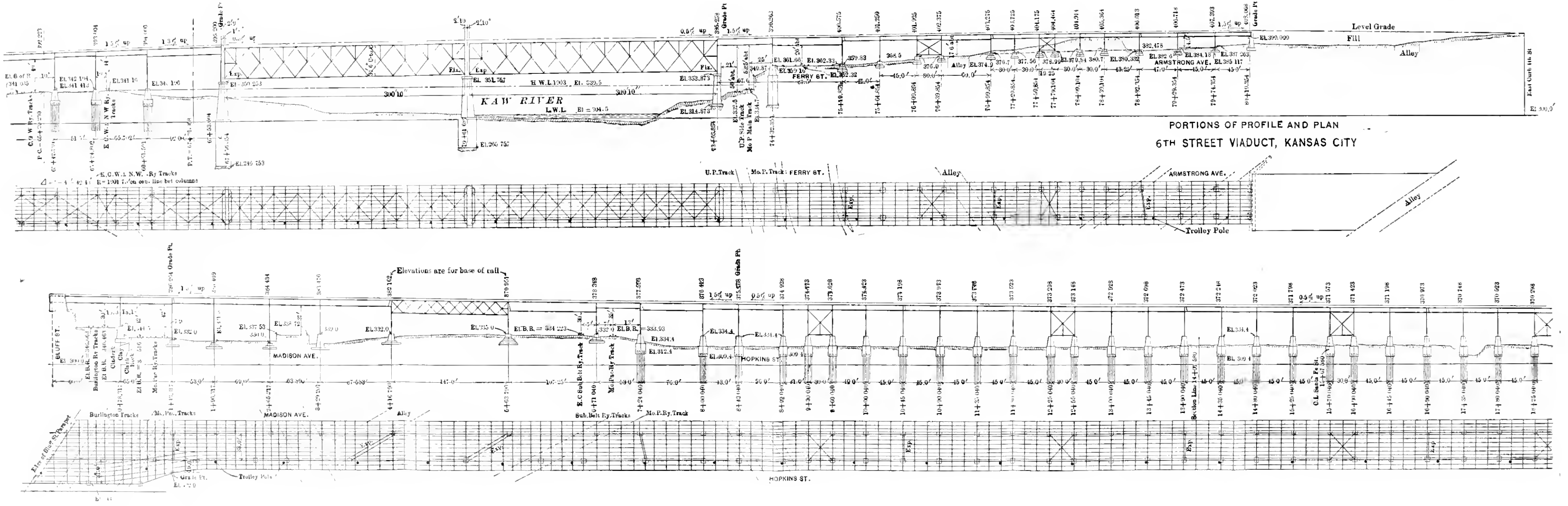
entrances to the bottoms; and, on the west side, giving access to Minnesota Ave., the principal business street of Kansas City, Kans., there is only one passageway. In addition to the steep grades to be contended with, the vehicular route to every freight station is over dangerous grade crossings frequently congested with traffic, while the interurban travel is compelled to follow circuitous routes.

Not only does the highway traffic to and from the bottoms operate under adverse conditions, but the street railway facilities are inadequate, there being only two through lines to handle the entire between-city travel. One of these is a very circuitous surface line, operated at great discomfort because of the many grade crossings, the steep grades, and the congestion of streets in the bottoms. The other line is elevated for a considerable part of the way, but the route is indirect. The need of additional rapid transit across the bottoms was long keenly felt by the street railway interests and the public, especially the latter.

Various measures have been devised from time to time to expedite this travel, and a viaduct has often been advocated. Shortly after the disastrous flood of 1903, which, by the inundation of the entire west bottoms, for a time completely cut off all intercommunication, a plan was proposed for the two cities to build jointly a street-car and vehicular viaduct, but it soon became apparent that no money was or would be available for such a purpose, and in any case the difficulties of two city governments jointly handling a structure located in two cities, two counties, and two States, seemed almost insuperable.

The proposition to build a toll viaduct by private capital for an investment was then broached, and investigation was undertaken to determine the feasibility of such a plan. A preliminary survey showed that a viaduct could be built across the accreted lands near the Missouri River, then unoccupied, which would shorten the distance from center to center of business districts by street car 4 150 ft. (26%) and 18 min. (60%); would give easy grades and uninterrupted passageway from city to city for highway travel; and, by a judicious location of laterals and approaches, could give convenient access, with elimination of all grade crossings, to each of the various freight stations in the bottoms.

Careful investigation, extending over a number of months, was then undertaken to determine: First, the amount of time to be saved by the patrons of a viaduct; second, the deterioration of equipment



PORTIONS OF PROFILE AND PLAN
6TH STREET VIADUCT, KANSAS CITY

to be lessened; third, the additional loads to be carried per vehicle; fourth, the actual money to be saved by the user; fifth, the amount of toll the user could afford to pay and still profit by the use of a viaduct; sixth, the probable volume of traffic to patronize such a structure; seventh, the effect of the viaduct on the distribution of population, and the consequent increase of travel; eighth, the probable growth of the cities and the increase of business; ninth, a minute and thorough estimate of the cost of construction; tenth, the annual cost of operation; eleventh, the annual income; and twelfth, the earnings to be realized.

For many of these questions there was no precedent, and some conclusions were based on unsatisfactory data. The determination of the number of vehicles passing to and from the bottoms required merely an extended series of actual counts, secured from day to day, principally by detectives of the Pinkerton service. These counts were made at various times from October, 1903, to June, 1904; and numerous independent tallies agreed closely with a total of about 12 000 vehicles per day to and from the bottoms.

As a further verification of these figures, statements were obtained from leading transfer men and large haulers as to the estimated use they would make of the viaduct, the total sum of which showed an entirely satisfactory figure.

A study of comparative statistics showed Kansas City to have exceptional business prospects, for, while at present twenty-second in population, it ranks as ninth in bank clearings, twelfth in total postal business, second in railroads, second in grain business, and it is situated in the center of a rich and productive territory.

Neither did the estimates of cost of construction and maintenance offer particular difficulty; but the determinations of the individual profit to be secured and the toll which could be reasonably expected were but approximations.

The matter of the utilization of the viaduct for street cars was taken up early with the company operating the cars in the city, and with certain suburban lines, with the result that tentative agreements, later duly confirmed by contracts, were made for traffic arrangements

The conclusion of the preliminary investigation was that the structure as then planned, reaching from city to city, with laterals at properly selected points giving uninterrupted access to the freight houses

was feasible as an investment, could be built for about \$2 275 000, could be operated for about \$180 000 per annum, and could earn from 5% upward on the capital stock, depending on the future growth of the city.

The Kansas City Viaduct and Terminal Railway Company was organized to build and operate such a viaduct, and the Common Councils of the two cities granted to the Company the necessary franchises. The principal provisions of these franchises are:

The franchise is to continue in force for thirty years;

A schedule of maximum tolls is fixed;

A clause provides that the City may at the end of ten years purchase the viaduct at any time, by notifying the Company one year in advance, at the actual cost and a stipulated varying interest rate;

A provision is made for the Company to pay the City 2% of its gross revenue.

As soon as certain necessary legislation was obtained from the Missouri Legislature, financial arrangements were completed, right of way was secured, and the construction, for which general contracts had already been made, was begun.

It had been the intention to commence the Mulberry Street Approach first, to finish that approach and the east end of the main structure from the approach to the eastern terminus, and have this portion in operation before the completion of the main viaduct; but the Company's attorneys discovered many difficulties in getting right of way for the approach, so the construction of the Main Viaduct was proceeded with until the legal complications could be untangled.

The following pages contain a discussion of the design and construction in the following order:

1.—Superstructure, design, manufacture, and inspection.

2.—Substructure, design, and construction.

3.—Engineering field methods.

4.—Erection of superstructure metal.

5.—Floors, concrete floor, paving, street-car deck, hand-rail, and lighting system.

6.—Approaches and laterals.

7.—Final costs.

1.—Superstructure.

The design provides a roadway 30 ft. wide over a part, and 38 ft. wide over the remainder of the structure, to carry a loading varying according to span from 50 to 100 lb. per sq. ft., or a 15-ton road roller; and, separated from the roadway by a substantial hand-rail, a double track for electric cars, to support a continuous line of cars on each track, each car weighing 40 tons and having a length of 43 ft. 3 in. over all. Each of these assumed loads is increased by an impact percentage as noted hereafter.

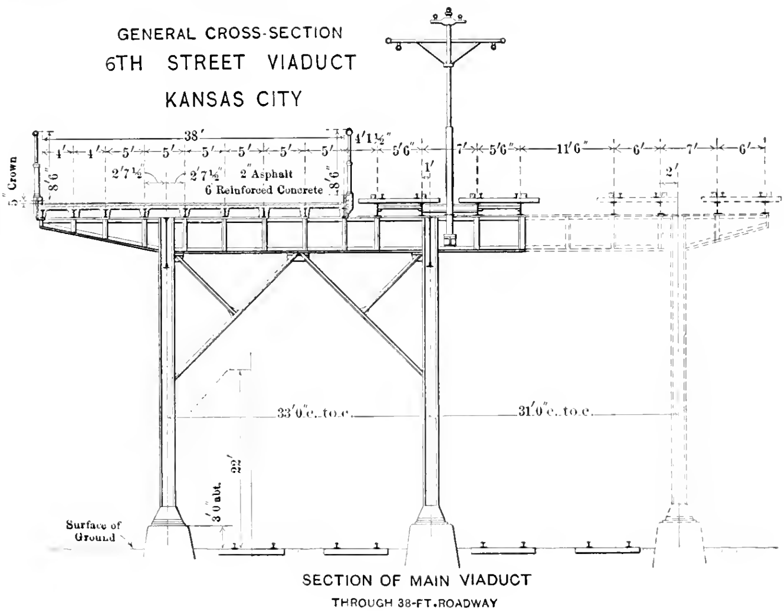


Fig. 2.

Before adopting the layout for the spans, an extended economic study was made by preparing estimates for different arrangements of girders and columns. The varying amount of shopwork, and the unit cost of material, as well as the total weight of metal per linear foot. were duly considered, the resultant variation of substructure cost with span length was estimated, and the most economical girder length. floor arrangement, and tower spacing for the confronting conditions were definitely determined.

This led to the selection of a two-column structure with one column

about under the center of the roadway, and the other about under the center of the motorway, with longitudinal girders riveted into the columns, cross-beams riveted into the girders and columns, and stringers laid on top of the cross-beams.

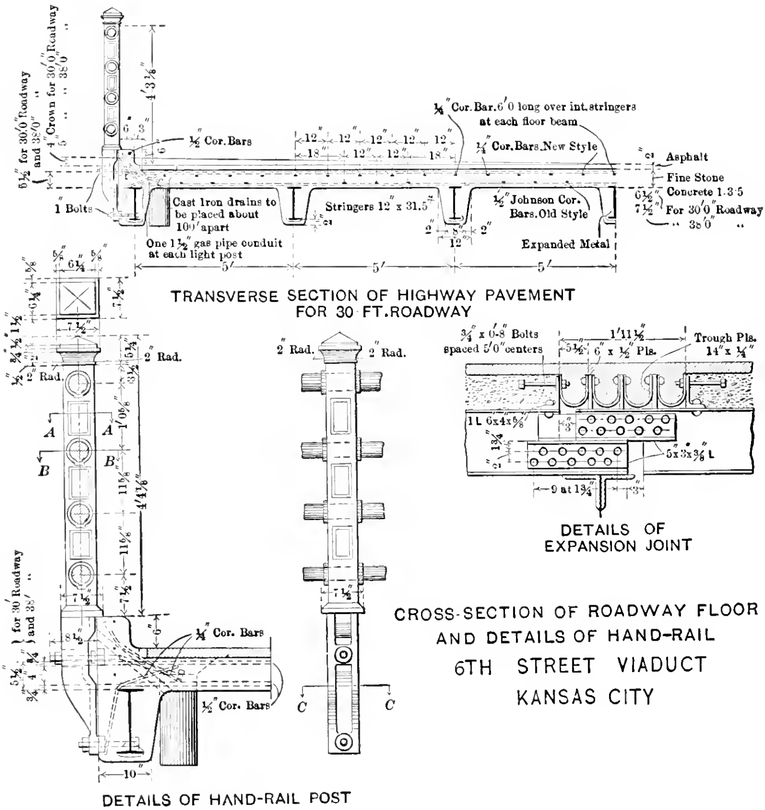


FIG. 3.

Considerations of possible increase of railway traffic resulted in the decision to arrange the viaduct so that at any time there could be added two additional tracks, each to carry a continuous line of cars, each car weighing 70 tons and having a length of 43 ft. over all. This is to be done by adding a third row of columns with girders, and splicing the cross-beams to the original motorway cantilever beams.

The two columns are 33 ft. from center to center transversely, and spaced so as to be loaded equally when the final loading is applied.

The third column will be 31 ft. from the original motorway column and at mid-point between the two added tracks. The transverse bracing is arranged so that tracks may be laid on the ground between the columns, and occasional turnouts are allowed for by spans of special length.

Longitudinally, the structure is divided into sections composed of one span of 30 ft., and seven spans of 45 ft. The 30-ft. spans have longitudinal bracing to care for the entire thrust of the 345-ft. section, thus imposing very few obstructions to free transverse passageway below, a stipulation of certain river-front property holders. The mid-girder between the towers is supported at one end in a pocket or shelf so as to slide and allow for expansion.

The highway floor is of concrete, enveloping the steel stringers. and supporting a pavement of asphalt. Hand-rails are placed on each side of the highway. The car tracks are of ordinary rails with the usual creosoted pine ties and guard-rails. The maximum grade on the main viaduct is 1.5%, and the minimum is 0.5%, this latter being maintained so as to drain the asphalt pavement properly.

The following working intensities were used in the design:

Tension Stresses.

	Pounds per square inch.
Flanges of plate girders (counting in one-eighth of the web)	14 000
Rolled shapes	16 000

Bending Stresses.

Extreme fiber of rolled sections	16 000
Extreme fiber of timber beams	2 000

Compression Stresses.

Top chords	$18\,000 - 70\,\frac{l}{r}$
Inclined end posts	$18\,000 - 80\,\frac{l}{r}$
Intermediate posts and diagonals	$16\,000 - 80\,\frac{l}{r}$
Lateral struts and bracing	$16\,000 - 80\,\frac{l}{r}$
Columns	$16\,000 - 60\,\frac{l}{r}$

Shearing Stresses.	Pounds per square inch.
Webs of plate girders	10 000
Rivets	10 000
Impact, for motorway loading	$I = \dfrac{400}{L + 500}$
Impact, for highway loading	$I = \dfrac{400}{L + 150}$

Under extreme temperatures and full live load, the shortest columns were computed to have an extreme fiber stress of 24 000 lb. per sq. in., but this was not considered excessive for such unusual conditions.

All calculations were based on the use of equivalent uniform live loads instead of actual wheel concentrations. Because of the very considerable labor involved in computing anew, for each slight change in panel length, the cross-beam reactions, the results from a series of panels of different lengths were plotted and a curve was drawn connecting all points. This diagram proved of great service in designing the many special girders, for a principal part of the calculations was saved by taking off directly the amount of the imposed loads.

Trestle Construction.

The various special girders and columns were detailed in conformity with the standard portions of the structure, and may be generally included in the following description.

Stringers.—Both the highway and the motorway stringers consist of I-beams set on top of the cross-girders and cantilevers, and connected thereto by rivets or bolts. Small shims are used under the highway stringers to provide a crown for the roadway. All stringers are continuous for the corresponding girder length, either 30 ft. or 45 ft., and are joined by small splice-plates. At expansion joints, stringers are halved and reinforced with bearing angles so that they slide upon one another. Connecting the motorway stringers, there is a rigid system of bracing, composed of single-angle diagonals, and of cross-struts of either one angle or one channel. The connections for all this bracing are made by lug-angles to the webs of the beams so as to leave their upper flanges clear for convenience in laying the ties.

Cross-Girders and Cantilevers.—Cross-girders are spaced at 15-ft. centers, thus placing two intermediate beams in the 45-ft. girder spans and one in the 30-ft. spans. They are riveted to the webs of the main girders or to the columns, with their upper flanges level with the tops

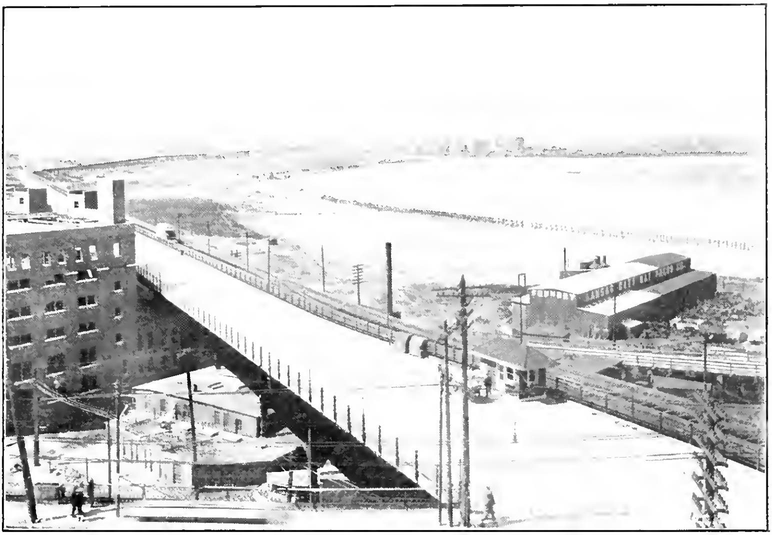


FIG. 1.—THE COMPLETED VIADUCT LOOKING FROM THE EASTERN END TOWARD KANSAS.

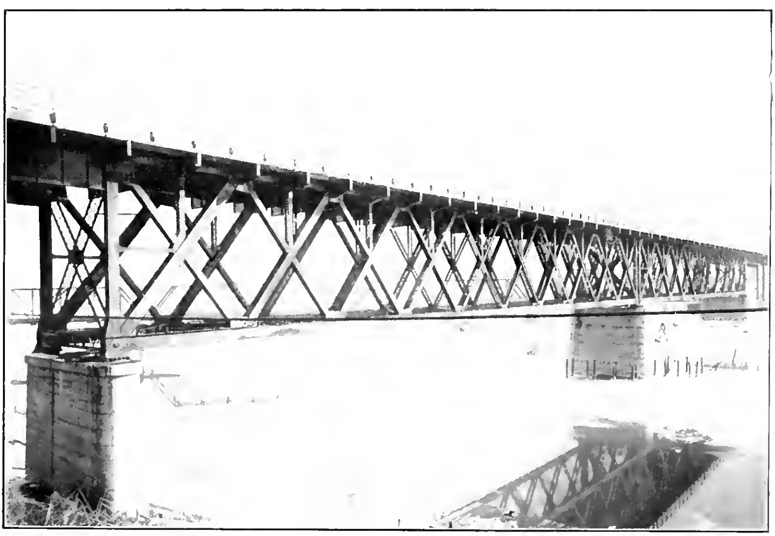


FIG. 2.—THE COMPLETED BRIDGE OVER THE KAW RIVER.

of the main girders. These beams are 52 in. deep, with ⅜-in. webs, and flanges composed of two angles. The intermediate stiffeners, which are pairs of angles placed at each concentration point, are crimped, while the end stiffeners have fillers on the web. The cantilevers at each end of each cross-beam follow the same detail, that on the highway side varying in depth from 52 to 12 in., though the motorway cantilever is of full depth to its end, where provision is made for splicing on a future cross-beam. Splice-plates extend over the girders or columns, and connect the top flanges of the cross-beams and cantilevers. At columns, the thrust at the lower flange is carried into the cross-beams with cast-iron thrust-blocks bearing against the channels of the posts and the webs of the main girders. These blocks were cast to a driving fit, were driven to place after the field riveting was completed, and were also supported by bolts through the column webs.

Main Girders.—The standard 45-ft. girders are 72 in. deep, with ⅜-in. webs and duplicate flanges of two angles and cover-plates. The general details are the same as for the cross-beams, except that the intermediate stiffeners are spaced 5 ft. apart.

Columns.—The columns, throughout, are of two rolled or built channels joined by a built I, with the webs of the channels in general parallel to the longitudinal axis of the structure. Stay-plates connect the flanges of the channels. Columns extend clear to the top of the main girders and cross-beams, providing for them full riveted connections. Lug-angles and brackets are riveted to the columns for the connection of the bracing, and shelf-angles are provided for convenience in girder erection. The shoe is built of side-plates and angles, fastened to the channels, and distributing the load to two heavy base-plates. Curved plates, 18 in. high, are fastened to each side, as sleeves for the anchor-bolts, and as bearings for the anchor-nuts.

At expansion points the columns have on one side pockets made of plates riveted to the channels supporting a bottom or base on which the girders slide. The tops of the columns are tied back to the adjacent span in a manner to relieve the main connection rivets of tension due to eccentric loading.

Bracing.—Lateral bracing is used only on curves, and in each tower span, it being considered that elsewhere the concrete floor would give all needful lateral rigidity. Where used, it consists of two diagonals of two angles each, with sufficient area to take the stress in either tension or compression. These angles are placed just below

the cross-beams, are riveted to the column beams and to the main girders, and also to each other beam they touch.

Longitudinal bracing at the towers is made up of two diagonals and a lower horizontal strut, all composed of four angles, laced, I-shaped, and of dimensions to fit into the columns. At the upper points the diagonals are attached also to the longitudinal girder.

Transverse stiffness is provided by knee-braces of two angles fastened to the column webs, and to the lower flanges of cross-girders, each column being braced in this way. The ends of pairs of girders resting in the expansion pockets are connected by a light bracing frame.

Truss Spans.

At the Kaw River are used two truss spans, each 300 ft. 10½ in. from center to center of end pins. They are of the double-intersection, deck, Warren type, with subdivided panels, there being ten main panels. The depth from center to center of chords is 37 ft. 6 in., and the trusses are spaced 33 ft. apart. These spans are riveted throughout, and attention is called to their exceptional size; each span weighs about 900 tons.

The floor system is arranged just as on the trestle, with cross-beams and cantilevers riveted into the top chords and short vertical posts. Two small expansion joints for the floor are used on each span to avoid the distortion of the floor-beams as the top chord shortens.

The top chords are of two web-plates, a cover-plate, and four angles, with the upper angles placed inside. The various top chords are spliced only enough to assure alignment, and the contact of abutting webs is relied on to carry the compression. The bottom chords are of two built channels laced with angles. The mid-span chord has an area of 212 sq. in.

Diagonals are alternately of built channels turned out, and of built I-struts, the latter passing through the former. Thus the channels rivet on the outside of the gusset-plates and the I's between them.

The upper laterals are of two-angle diagonals placed in the plane of the lower edge of the chord. They reach the chords only at main panel points, and therefore intersect at the intermediate floor-beams, to which they are attached by plates passing through slotted holes in the webs, and are connected thereto by lug-angles. The lower laterals consist of diagonals, between main panel points, of laced, four-angle, I-struts, the depth of the chords.

The upper laterals are made heavy enough to dispense with inter-

mediate transverse bracing between the trusses. Between the end posts there is cross-bracing of four-angle I-struts from the floor-beam above to a transverse strut below. This arrangement avoids the transference of loads from truss to truss due to unequal deflections, and also the unsatisfactory details of transverse bracing fastened to other than vertical members.

The shoes are built up of plates and angles, and are connected to the trusses by 12-in. pins. Ten segmental rollers, 8 in. high, support the expansion shoes, and cast-steel base-plates are used at each end.

The trusses were calculated on the assumption of all loads being equally distributed between the two systems. Although the trusses are equally loaded when fully loaded, the resultant stresses vary somewhat, because the larger part of the load on the highway truss is from the concrete floor, while on the motorway truss the live load is much greater. Both were calculated, and, in the make-up of sections, the chord stresses of the highway truss were combined with the web stresses of the railway truss and the two trusses were made identical.

The one other truss bridge of the viaduct is a skewed span, 147 ft. long, over railway tracks. This span is of the same general type as that at the Kaw River, except that the intermediate vertical posts are absent, the floor-beams being supported on the gusset-plates. The depth of trusses is 15 ft. 6 in. At each end, owing to the skew, there is a diagonal girder, 62 ft. long, and the regular cross-beams are riveted into it. The span is supported on shoes of the usual type, that at the roller end resting on seven segmental rollers supported on a cast-steel base. There is no bracing, either longitudinal or transverse, in the columns under this span, and especially heavy sections were used. The columns supporting the fixed end are 42 by 32 in., having about 200 sq. in. of section. Each of the four columns is anchored to its concrete pedestal by eight $2\frac{1}{4}$-in. bolts 9 ft. long.

Specifications and Manufacture.—The consulting engineers prepared complete detailed drawings, and from these the contractor prepared working drawings. These engineers' drawings do not give in full all minor details, such as the exact spacing of rivets, sizes of small plates, etc., but do fix the dimensions of all members and of all parts thereof, allowing the shops to fill in the lacking dimensions according to their preference. The shop drawings were then checked by the consulting engineers, who made the necessary corrections, and finally approved all details. As a representative, located at the shops,

gave all points prompt attention and decision, much time was saved and correspondence avoided.

All metal is open-hearth, medium steel. The specifications provided that all plates should be rolled from slabs of at least six times the plate thickness, which slabs were to be made by rolling an ingot and cutting off the scrap. The following maximum percentages of ingredients were fixed:

Phosphorus, 0.04 to 0.06; sulphur, 0.04; silicon, 0.04; manganese, 0.70. The ultimate tensile strength on test pieces was to be from 60 000 to 70 000 lb., and the least allowable elastic limit, 35 000 lb. The least allowable elongations on specimens varied from 20 to 24%, with a corresponding reduction of area of from 44 to 36 per cent.

First-class workmanship in every respect was specified. Sheared or hot-cut edges of plates were planed so as to remove $\frac{1}{4}$ in. of metal. Rivet holes were sub-punched $\frac{1}{8}$ in. less, and reamed to a diameter $\frac{1}{16}$ in. greater than that of the rivet, and it was required that the reamed holes be perpendicular to the metal surface. A modification in part of the foregoing was made, and the field holes for the stringer bracing and certain other minor parts were punched full size.

The ends of all girders and abutting members were planed to make perfect contact. Stiffeners on girders were ground so as to fit tightly against the flanges. This was strictly enforced in the case of stiffeners carrying concentrated loads, and in certain cases where the stiffeners did not quite touch they were removed and new angles riveted on. Edges of spliced web plates were planed and put in contact for their full length.

It was specified that special attention be given to the cleaning and painting of the metal-work in the shop, where one coat of paint was applied. The metal was cleaned of rust, dirt, and loose scale before painting, and all surfaces of metal in contact were required to be painted before being put together. In a few instances it was necessary to have finished work cut apart in order to do this painting.

A large portion of the work was laid out direct, without the use of templates. Gauge punches were used almost exclusively on pieces which duplicated several times. There was much duplication, of which could be mentioned 220 45-ft. girders, 480 floor-beams, 350 cantilever-beams, practically all stringers, and so forth. Although the columns are all of different lengths, groups were made together requiring only the details near the lower ends to be varied.

The rate of shop work varied considerably, but averaged about 900 tons per month, finished and shipped. The Kaw Bridge spans, comprising some 1 800 tons, were fabricated and shipped in five weeks.

Careful inspection was made of all operations in the shop. Rivets were examined as driven; there was a general supervision of laying out, punching, shearing, assembling, etc.; and the finished piece was checked before loading. The result of this careful inspection was apparent in the very few errors found in the field.

Full records of the progress of each piece were kept, and weekly reports were made giving the status of work of all members and the percentage of the estimated total completed. The weights were all calculated from the drawings, and checked against the scale weights.

2.—Design and Construction of Substructure.

Surveys and borings showed bed-rock to be about 50 ft. below the ground surface across the entire valley except for a few hundred feet from each bluff, where it is very much lower. The material overlying bed-rock was found to vary from soft muck and running sand to clay, firm sand, and gravel.

Because of the uncertain bearing capacity of this accreted land, and of a possibility of scour, piles were used under 276 of the 326 pedestals of the viaduct.

The first design contemplated the use throughout of a patented form of concrete pile, and the pedestals were designed accordingly. This patented pile was to have been built by driving into the ground a steel shell or mould having a removable point, and on reaching the required depth the shell was to be filled with concrete and gradually pulled out as the concrete was tamped into place. Further consideration and examination of driven piles made it seem to be undesirable and inexpedient to use this patented pile in the wet soil to be encountered.

The manifest desirability of a concrete pile made the engineers loath to abandon that type of foundation, and led to the adoption of a built-up driven pile. A number of experimental built-up piles were made at once, and, after the feasibility of driving them was demonstrated, their general construction was immediately undertaken.

Of the pedestals not supported on piles, some rest on rock, some on hard clay, and some on soft material. Thirteen pedestals near the east end of the viaduct are under the buildings of a machine shop or under

railroad tracks. The material here is very soft, and, after experiments, an intensity on the soil of 1 ton per sq. ft. was allowed. In order to have the excavations shallow, the bases were reinforced. Plain steel bars were used.

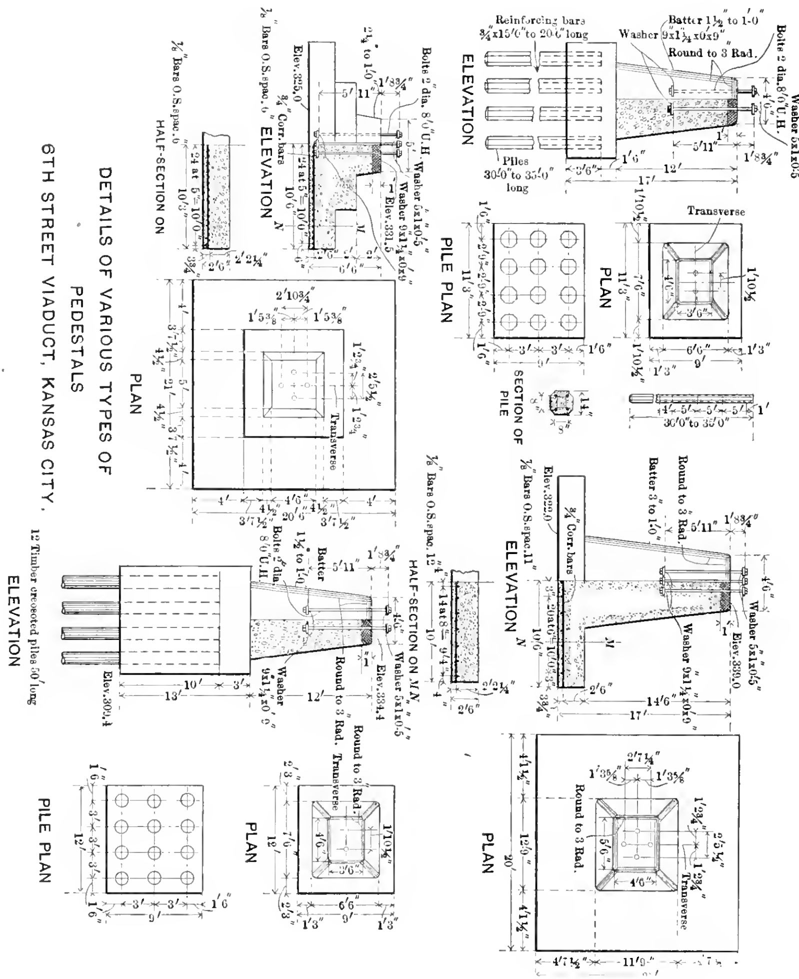


FIG. 4.

Near the western end of the viaduct the pedestals rest on firm red clay, loading it to about 3 tons per sq. ft. The three Kaw River piers are founded on rock, two being sunk by the pneumatic process, and one built by open excavation.

FIG. 1. CONCRETE PILES: FINISHED PILES CURING.

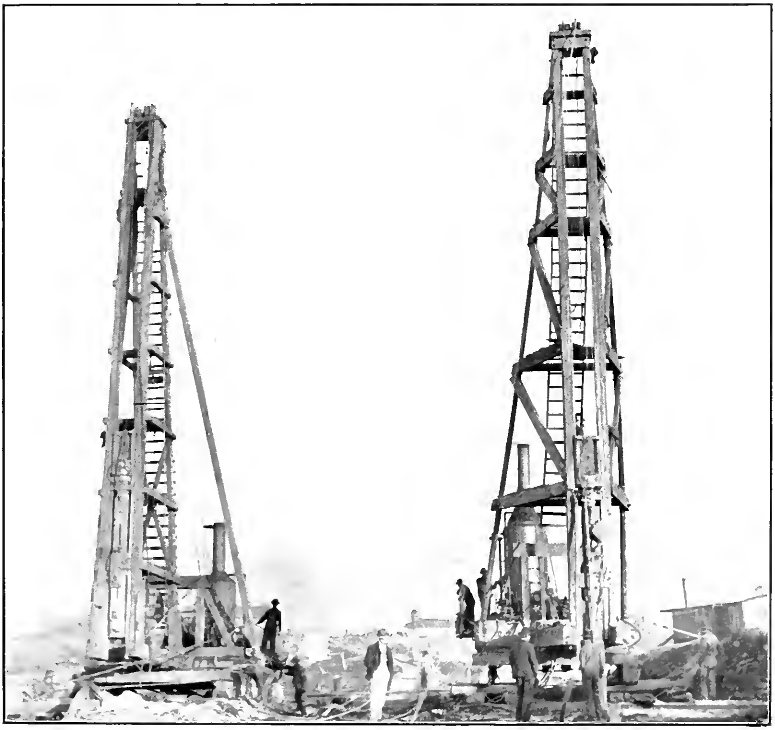


FIG. 2.—PILE-DRIVERS, SHOWING SPECIAL ARRANGEMENT OF FOLLOWERS.

Piles.—The moulded concrete piles are all of one size, 30 ft. long, 10 in. square at the lower, and 16 in. square at the upper end, with a diagonal 3-in. cut off each corner. Each is reinforced by four ⅞-in. steel bars, extending from end to end, symmetrically placed as near the surface as practicable, and surrounded by a cylinder of heavy woven-wire fencing intended to occupy a position about 1½ in. from the surface of the concrete. In the later piles the wire was omitted.

The forms or moulds for the piles were simple boxes, with bottom, sides, and ends of 2-in. plank suitably cross-braced. Lugs on the cross-braces engaged wooden wedges which tightened and held the parts together yet readily permitted them to be knocked down for moving. The diagonal corner moulding made the form almost water-tight. The upper end of the pile was shaped into a round head to facilitate driving, and the lower end was bluntly pointed. The several parts, of course, were made interchangeable. The interior surfaces were coated with oil before concreting. The piles were moulded lying on the ground, in groups of from 8 to 16, in position convenient to their point of driving.

The specifications provided that the piles be made of concrete composed of "1 part of Portland cement, 3 parts of Kaw River sand, and 5 parts of clean hard broken stone to pass a 1-in. iron ring". The cement used was Iola Portland, and cost, f. o. b. Kansas City, $1.60 per barrel. The sand of the Kaw River is very clean, sharp, and well graded, testing about 52% retained on a No. 30 sieve; its cost, delivered, was 75 cents per cu. yd. The stone was from quarries about the city, and cost $1.30 per cu. yd. delivered. There was considerable trouble in securing clean stone, for the limestone strata of this region are overlaid with clay, and, especially in a rainy season, it is impossible to prevent some clay from intermingling with the stone in the crusher bins.

Some limited experiments were undertaken to determine whether the amount of clay present had a deleterious effect on the concrete. Test bars made of measured quantities of material, mixed in a uniform manner, were prepared with the stone as it was delivered from the crusher, and with stone carefully and thoroughly washed and cleaned. When these bars were tested and compared it was found in all cases that the unwashed clayey stone made stronger concrete than the clean washed stone.

The concrete for the piles was made by hand on tight movable platforms. The practice was to mix together the stone, sand, and cement dry, and then add enough water to give a reasonably wet mixture. The material was distributed by wheel-barrows, and dumped into the prepared forms. After a 2-in. layer of concrete had been spread in the form, the reinforcing of wire and rods, already prepared, was laid in position, the remainder of the concrete was dumped in, tamped to place, and smoothed on top to a reasonably even surface. Sticks were used to tamp through the meshes of the wire and to hold the rods apart. The rods had a tendency to bunch together, and it required constant care and watchfulness to get them where they belonged. Several methods were tried to obtain the desired results easily, but none was entirely satisfactory. The conclusion is, however, that stiff iron spacers, either as a ring or as an X, at intervals of about 3 ft. along the rods, would give superior results.

Each pile was marked with the date made, and for several days thereafter was showered copiously with water. The side and end forms were usually removed in 48 hours, but the pile was not rolled from its base for about a week.

Over-anxiety to re-use the base forms without delay caused the loss of several piles, for the heavy green piles were easily cracked by very small inequalities of the ground. Weather conditions, of course, affected the rapidity with which the piles hardened, but they were generally allowed to cure from 3 to 5 weeks before use, although some were driven 2 weeks after being made. Each pile contains 1.4 cu. yd. of concrete. The finished piles are shown in Fig. 1, Plate III.

For driving, or rather sinking, the piles, heavy turn-table drivers, equipped with a special kind of follower, were used. This follower was of timber, about 12 ft. long, with guides to fit the leads at its upper end and mid-point, and with a deep cylindrical collar at its lower end. A standard 3 000-lb. hammer rested on the follower and was fastened loosely to it. The usual "pile line" was carried down, through a steel sheave, then up and made fast to the top of the leads. To this sheave was fastened a 10-ft. length of steel cable having a slip-noose at its lower end, and a "pull-off line" attached to the slip-loop of the noose. A water-jet pipe, consisting of a 2½-in. pipe 35 ft. long, with its lower end drawn down to a nozzle, completed the equipment. This jet pipe was handled by a line to a spool of the engine, and was supplied with

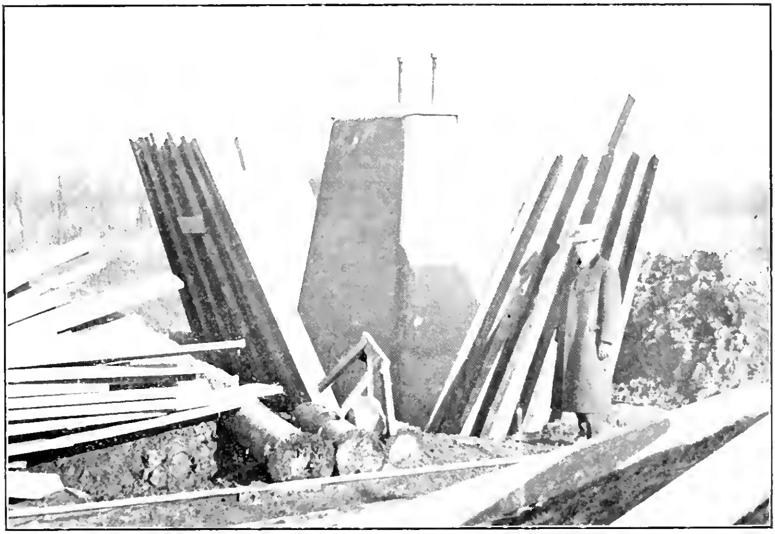


FIG. 1.—PEDESTALS: FORM REMOVED.

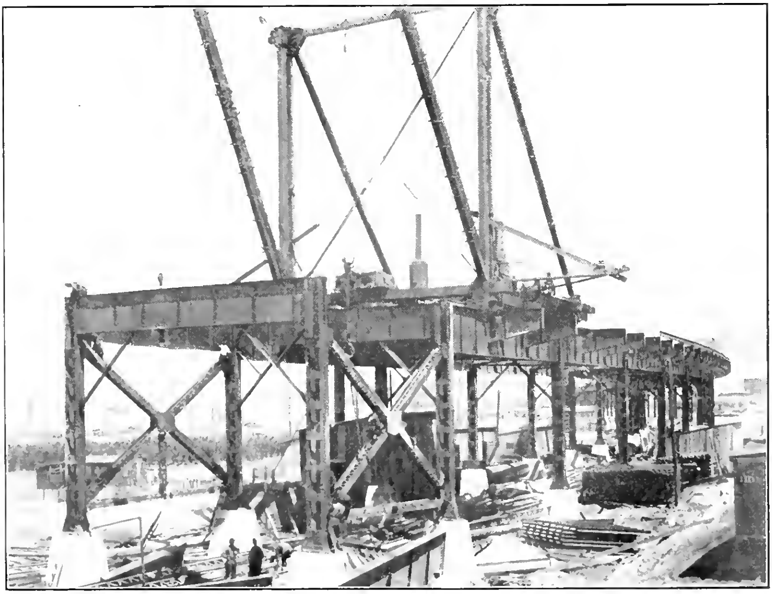


FIG. 2.—TOWER SPAN, WITH LONGITUDINAL BRACING.

water at a pressure of about 120 lb. through a 3-in. standard fire-hose. To provide the water for the jets, a pumping plant, consisting of a high-duty 18 by 10 by 12-in. fire pump with suitable boiler equipment, was erected on the bank of the Missouri River about $\frac{1}{8}$ mile from the structure, and a pipe line, from 3 to 4 in. in diameter, was laid from it along the entire viaduct. Fig. 2, Plate III, shows the pile-drivers and the special arrangements of the followers.

In general, piles were driven before any excavating was done, and they were sunk so that their upper ends were from 1 to 5 ft. below the ground surface, in the following manner:

After the driver had been set to the correct position by the adjustment of the rollers on which it stood, the hammer and follower were lifted to the top of the leads. The jet was then played on the desired position of the pile, and, as the ground softened, it was gradually pushed down full length. In the meantime the slip-noose cable was placed around a pile some 4 ft. from its upper end and the pile was raised and swung into the leads where it was straightened up and the follower settled carefully on the protruding head. By means of the sheave-line, the pile was then churned up and down, the pile, the timber follower, and the hammer resting thereon all going up and down together, while the jet was kept in constant service. Drops of as much as 6 or 7 ft. were made, without apparent injury to the pile, and the churning process was continued until the required depth was secured. By the "pull-off" cable, the slip-noose was easily loosened and removed.

The effect of running the jet down first was to cause the pile to keep its correct position, as it was found that otherwise each pile had a tendency to travel toward the adjacent ones previously driven, where the ground was still soft; and even then the pile tended to lean toward the side where the jet was played during sinking. It is believed that more accurate sinking could have been done with a pair of jets, one on each side. The simple slip-noose for handling the piles was developed only after the failure of several more or less elaborate schemes devised for the same purpose.

The time consumed in the actual sinking of a pile varied somewhat, but, where no particular obstructions were encountered, from 2 to 5 min. sufficed. However, the highest record for one gang in 10 hours was 36 piles.

It was soon apparent that, owing to the delay in starting, cold

weather would probably set in and stop concreting before all the concrete piles could be made, and it was necessary to devise a substitute so that the structure might be finished in the time assigned. Creosoted pine piles were, with reluctancy, decided upon, a rush order was given to pile contractors and creosoting plants, and, by constant attention, the piles were rapidly delivered.

The specifications called for:

"Piles cut from live, straight, long-leaf, yellow-pine timber; to be free from cracks or wind-shakes; to be so straight as to be at no point more than one-third of their sectional diameter out of a straight line joining the centers of the ends; to be of even, gradual taper from end to end; and to be not less than 10 in. in diameter at the small end."

The specifications for creosoting provided, after the usual requirements for dressing, peeling, and artificial seasoning, that:

"After the seasoning is entirely finished, pure, unadulterated dead oil of coal-tar shall be injected into the timber so that to each cubic foot of timber will be 12 lb. of creosote oil."

Additional equipment for handling the timber piles was hastily provided. The drivers were of the ordinary light timber-framed type, braced by guy lines to the top of the leads, which were 60 ft. high. Each was mounted on two wooden rollers, 1 ft. in diameter and 20 ft. long, resting on timber sills laid on the ground. Lateral movement was secured by sliding the driver along the rollers. This roller arrangement proved good, permitting rapid moving and setting. Both drop-hammers and steam-hammers were used, the latter doing much more efficient work. Water-jets, arranged about as previously described, were used. The efficiency of the jets was demonstrated in several instances when, because of a break-down of the pumps, attempts were made to drive without water, for these invariably resulted in the piles being injured and broomed, before they had reached their full depth, thus precluding further driving.

On a portion of the structure, 50-ft. piles were driven, reaching to bed-rock; on the remainder, 35-ft. piles were used, resting, as do the concrete piles, on gravel and on clay.

To determine the bearing capacity of a concrete pile, a test load was placed on one of the first piles sunk. The pile loaded was of standard size, was made on August 28th, driven on September 15th in 25 min., and the loading was begun on September 24th. To support

the load, a timber platform, 12 by 4 ft., was balanced on a single post resting on the pile head. Observations were made with an engineer's level, reading on a scale fastened to a piece of 1-in. pipe which was set into the pile a few inches and projected above the platform without being in contact with it. The load was of pig iron, each pig weighing about 66 lb., and was placed by hand at an average rate of 6 tons per hour.

During the first $1\frac{3}{4}$ hours the load increased from 4 tons (the weight of the platform) to 19 tons, with a very small settlement. This load was constant for the next $15\frac{1}{2}$ hours, and the pile settled 0.012 ft. During the next 4 hours the load increased from 19 to 35 tons, with an additional settlement of 0.038 ft. After an interval of $1\frac{1}{2}$ hours, loading was resumed for 4 hours and 40 min., with a settlement of 0.097 ft. During the 17 hours and 40 min. this total load was kept on the pile, there was a further settlement of 0.062 ft. In the $4\frac{1}{2}$ hours of unloading, the load on the pile was reduced from 43.75 to 4 tons, and the pile rose 0.021 ft., leaving a net settlement of 0.198 ft.

These results made necessary some revision in the pedestal design, as very much greater capacity had been claimed for the patented pile. Moreover, the material varied so much from place to place that this test was not believed to be representative of the entire structure, and some ready method for tests was desirable. It was clearly impracticable to load each pile to determine its bearing capacity, and as there is no way to estimate the capacity of piles sunk by jet, the following plan was adopted.

Creosoted pine piles, 40 ft. long, were furnished and called "test piles." In cases where, by reason of special softness of the ground in driving adjacent pedestals, or from the data of the borings, there was uncertainty, one or two of these "test piles" were substituted for the concrete piles. These were driven by using the drop-hammer in the ordinary way, without a jet, and before any jetting had been done in the immediate vicinity. Observations were made of the drop of the hammer and the penetration of the pile from 25 to 30 ft. down, or about the position to be occupied by the concrete piles, and the resultant bearing capacity was calculated by the well-known formulas. In all cases these results gave a greater safe load than that actually on the piles, in fact, it was usually necessary to resort to the jet to get the timber piles to the full depth.

In the main viaduct there are 798 concrete piles, a total of 23 950 lin. ft. in place; and 2 573 creosoted timber piles, aggregating 93 800 lin. ft. in place; a total of all piles of 117 750 lin. ft.

Excavation and Concreting.—The excavation for pedestal bases was comparatively simple, and the work was done by hand. In nearly all instances where concrete and the shorter timber piles were used, no sheeting or bracing was needed, the sandy, clayey material standing to depths of 8 or 9 ft. For about 80 pedestals, most of which required pits some 13 ft. deep in very boggy, sticky ground, sheet-piles of 2-in. plank were driven by hand around timber frames, and these frames were dropped down to brace the planks as the excavation proceeded.

As soon as the piles were sufficiently exposed they were sawn off at the required elevation. Before concreting they were cleaned of mud and filth by scraping with a hoe having a curved blade. Where concrete piles were not settled to full depth they were cut off with chisels. and the reinforcing bars bent over.

A number of pedestals, particularly those at the east end, under buildings and near tracks, required special attention in excavation with regard to bracing and shoring. One building, under which several were constructed, having been through two fires and a flood, was in a precarious condition, especially as it rested on soft flowing material. The base of one pedestal under it extends under the bed of an engine which was kept in operation by supporting it temporarily until the concreting was completed. For certain railway tracks carrying a heavy and frequent traffic, it was necessary to drive temporary trestles to permit the construction of adjacent pedestals.

Forms.—In most cases no forms were used for the bases of pedestals. the pits being merely filled with concrete to the required elevation. The pedestal shafts are practically of uniform dimensions, and the forms were devised for repeated use. Each side was built as a single piece, of 1-in. horizontal, matched, dressed boards, with vertical posts of 6-in. square timbers. The four sides were held together by three square frames or hoops of 3-in. plank, bolted at each corner, bearing sidewise against the studs. Wedges were driven behind all posts not properly bearing. These shaft forms were easily adjusted to correct position by the insertion of small wedges under the bottom. Sheet-iron corner moulds were used to give the concrete a curved corner of 3 in. radius. For the vertical edges, probably the best results were secured

by using sheet iron of No. 12 or 14 gauge, rolled to approximate radius and then nailed firmly to one side of the form and tacked to the adjacent side only enough to keep the concrete from getting behind. No particular harm was done to this corner mould in pulling down the form. The upper corners were shaped by sheet-iron or wooden moulds, or by the use of a curved trowel. The latter, in the hands of a skilled workman, gave excellent results, but its use was deprecated because of apparent desire to use unskilled labor. Two wooden templates for supporting and holding in place the anchor-bolts, all of which were placed as the concreting proceeded, were fastened to the form.

Concrete.—The concrete was specified to be mixed by machinery in the proportions: One part of Portland cement, three parts of clean, coarse, sharp sand, and five parts of hard, clean, broken stone to pass a $2\frac{1}{2}$-in. iron ring; the proportions to be determined by volume; all ingredients to be measured loose; and enough water to be used to form a wet concrete.

In addition to the usual requirements for package delivery, etc., the specifications for the cement were: To be so fine that 97% in weight will pass a No. 74 sieve, and 90% pass a No. 100 sieve; to be of such tensile strength that briquettes of neat cement will show, for 1 day, from 150 to 250 lb. per sq. in.; for 28 days, from 400 to 600 lb. per sq. in.; to show no drop in strength; to be slow-setting; preliminary set in not less than 30 min., final set in not less than 3 hours; to undergo boiling and steaming tests without failure.

Sand was defined as "particles of hard, clean stone which will pass a $\frac{1}{4}$-in. mesh sieve, and not less than 50% of which shall be retained on a No. 30 sieve." Both the Kaw and Missouri Rivers have great supplies of excellent sand, that of the former being somewhat cleaner, but both coming within the requirements.

"Sound, hard rock, free from all dust and dirt," was called for, but, as has been noted, the output of local crushers, containing more or less clay, was used. It is difficult to state just how much clay was allowed to be in the stone, as a given load would seem very muddy when wet, yet clean when dry. The stone was generally accepted when there were no lumps of clay, or of clay and stone chips, even though many of the stones were muddy. Inspectors were stationed at the quarries to examine the material as loaded, yet, in spite of all care, some stone was used which contained small balls of clay.

Special detailed specifications in regard to mixing concrete were distributed among the various sub-contractors, to assure uniformity of product.

The concrete was mixed so wet that no real "tamping" could be done, but it was deposited approximately in layers of from 9 to 15 in., and "thoroughly spaded" by thrusting a spade to the full depth of the blade at intervals over the entire area, special attention being given to spading near the surfaces of piles and forms, so that the mortar would have perfect contact. It was in general handled from the mixers in wheel-barrows, either on the ground or on elevated runs, as conditions required, and was thrown freely from the tops of the pits or forms. For other pedestals and for certain abutments, slightly more equipment was needed, and derricks with boxes of simple well-known types were used in handling materials. Anchor-bolts were usually placed after the lower half of the shaft of the pedestal had set, and they were very carefully adjusted to exact position. The upper foot of each pedestal was of rich concrete made with Joplin flint or granite chips. One of the pedestals, with the form removed, is shown in Fig. 1, Plate IV. The pedestals were finished from $\frac{1}{2}$ to $\frac{3}{4}$ in. low, to allow for grouting under the shoes.

Various types of mechanical concrete mixers were used, both batch and continuous machines. The continuous mixers automatically measured the sand, stone, and cement by means of worms, and were mounted so as to be easily portable. Although considerable pedestal concrete was made with these machines, the results were not as satisfactory as those produced by rotary batch mixers which mixed the concrete in revolving drums or boxes. The output of the continuous mixers was not uniform, even though they measured the materials accurately, especially where the vicissitudes of the work necessitated intermittent operation, with frequent stops and starts; the water was not easily regulated, and the mixture was not homogeneous. Moreover, after a time, numerous spots appeared on the surface of the concrete from these machines. It was found that these were caused by small balls of clay, from $\frac{1}{8}$ to $\frac{3}{4}$ in. diameter, embedded in the concrete, the results, of course, of clayey stone. Careful examination, however, proved that these clay balls were practically absent in the work of the batch mixers, showing that the churning and tumbling had been sufficiently vigorous to pulverize and distribute all lumps of clay, with consequent harmless-

ness; while the mixing of the continuous machines had not been sufficiently vigorous to do so and therefore was inferior.

Certain pedestals, abutments, and retaining walls had special forms, made of 2 by 6-in. matched, dressed flooring, laid on vertical studs spaced from 2 to 3½ ft. from center to center. One side of each form was braced to the ground, and the other side was adjusted to it by numerous malleable-wire ties. A strut, fixing the correct width, was placed near each tie-wire.

Vertical cleavage planes were made in abutments and retaining walls, separating those parts of the wall having different load intensities on the foundation. A cross-partition was built in the form, and the concrete was finished against it. Before building the adjoining part of the wall, the partition was removed, and tar roofing paper was nailed securely to the exposed surface, thus effectually preventing cohesion of the fresh material to the old.

Kaw River Piers.—The three piers of the Kaw River Bridge are of concrete, of the same materials and proportions as used for the pedestals.

The west shore pier rests on a ledge of rock about at low-water level, so that the construction was very simple, and the work was done in the dry. Light sheet-piling was driven around the pit, which was excavated by hand. Hand-pumps were sufficient to take away the seepage water, and the bed-rock was easily exposed, so that its surface could be cleaned, washed and prepared for the concrete. The base of the pier was built up nearly to the ground surface.

The other two piers also rest on bed-rock, and both were sunk by using pneumatic caissons. The caissons were similar; that of the channel pier was built on shore and floated to place, and that of the east shore pier was built in place.

The caissons were of timber and had steel cutting edges. Timbers, 12 in. square, of Oregon fir and Southern pine were used; and were thoroughly bolted and drifted together. The walls of the chamber were 3 ft. thick, and the roof was 6 ft. thick. Every other stick of the upper course of roof timbers was omitted, to give bond for the concrete.

The outer timbers were vertical, and each was fastened by at least two bolts extending into the chamber, besides drifts. Horizontal timbers were lapped at the corners, log-house fashion, thus avoiding extensive framing. The chamber was sheeted inside with 3-in. planks,

beveled for caulking, and fastened with boat spikes driven through bored holes. Every bolt, and the shank of every spike and nail projecting into the chamber was carefully wrapped with oakum before being driven entirely home or finally tightened up. On the outside all bolt heads were countersunk, thus leaving a smooth surface. Two cross-bracing trusses, with adjustable tie-rods, stiffened the walls of the chamber.

A false bottom was put in the caisson for the channel pier, in order to float it to position; and, for this pier, a light tramway, on two pile bents supporting a standard gauge track, was built from shore to pier site for conveying materials. Concrete was mixed on shore and carried out on push-cars in skip boxes. A derrick barge handled the boxes and the timbers for the crib.

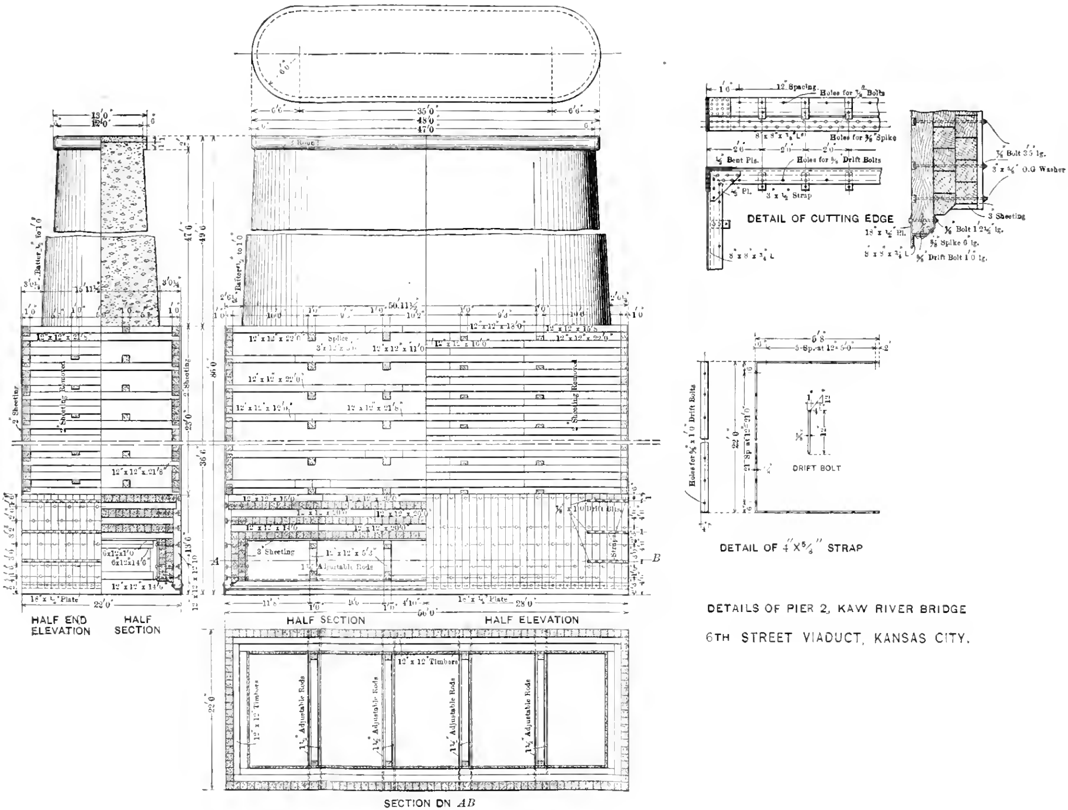
The crib, above the caisson proper, was of one thickness of 12-in. timbers sheeted outside with 2-in. plank nailed on vertically.

The usual man and material shafts, with air pipes, blow-out pipes. etc., were installed. The locks used were very simple, being made by merely putting cast-iron diaphrams with drop-doors at each end of a section of ordinary shaft. The one objection to locks of this kind is the necessity of an inside lock tender, and this is not entirely a fault, as men are much less likely to be "plugged" by a lock tender with them in the lock than by a man outside who operates without definite knowledge of conditions inside.

The material excavated was sand, with occasional deposits of silt, and very many boulders, so that most of the material was blown out dry. Numerous relics of the great flood were found in the river bed, indicating deep scour. The metal, notably some street car rails and some parts of steel bridges, was carried down and embedded in the concrete.

Considerable blasting was necessary to prepare the bed-rock for piers. A ditch 3 ft. wide was blasted out around the entire perimeter of the cutting edge, so that the concrete is at no place less than 2.5 ft. below the bed-rock surface. The middle portion of the rock was not removed, but was washed clean, as also was the entire inside of the chamber.

Concrete for filling the chamber was of standard proportions, and was mixed outside and locked into the pier through material shafts 2 ft. in diameter. It was placed in the dry, and carefully tamped to place.



DETAILS OF PIER 2, KAW RIVER BRIDGE

6TH STREET VIADUCT, KANSAS CITY.

especially against the ceiling, the boulders and pebbles carried down being mixed in. When the remaining space had been reduced to such small dimensions that the men could no longer work, the air was cut off, all doors were allowed to swing open, and into all shafts a thin grout was quickly poured from above until it was some feet above their lower ends. Ordinary concrete was deposited to fill them.

Coffer-dams were not used, but the shafts were built up as soon as the crib was of the estimated required height, and from this point down the sinking was handled very carefully so as to keep the shafts in correct position. For the last 16 ft. of sinking for Caisson No. 2 the shaft moved out of position only ½ in. at each end, and that in opposite directions.

The sinking of Caisson No. 1 was complicated by the presence of a 30-in. high-service water main lying so close to the pier site that the caisson had to be rotated to clear it. Pile bents were driven under the pipe for a support, but these went down, and a sling was put around it and passed up over an **A**-frame to a distant anchorage. All attempts to rotate this caisson to its correct position after it was below the obstruction failed.

These caissons were models in construction; they were well lighted by electricity, and constant and adequate air pressure was maintained. Natural gas was available for fuel.

For Pier 3 the shaft form was of 2 by 6-in. matched dressed flooring, which was placed horizontally on the sides of the pier and fastened to vertical studs, but on the curved ends it was placed vertically and drawn into position by ⅝-in. round iron hoops. These hoops were passed through holes in the vertical studs at tangential points and drawn tight by nuts threaded on their ends. Interior circular templates were placed near the bottom and at the top of each 10-ft. section of the end staves to hold them to position, but were removed as the concrete was built up. Ties of heavy malleable wire were put across between pairs of studs, and twisted tight to bring the sides to proper place. These ties were placed with such frequency as to hamper the placing of concrete, but could not be dispensed with. The building of a 10-ft. section of form and of filling it with concrete alternated, so that the pier was stripped below as it was built up above. Forms of this kind are not considered entirely satisfactory, being easily warped and distorted, and often delaying the concreting for readjustment.

The forms for the other two piers were made of the same planking, in 8-ft. lengths, but this was placed vertically all around the pier. Rigid frames, 4 in. thick, with curved ends of built-up plank nailed and bolted together, were spaced 4 ft. apart, and supported the sheeting. Five ½-in. tie-rods held the sides of the frame together, and cross-bracing at each end and on each side provided against lateral displacement.

The lower edges of the copings were formed to the required radius by sheet-iron corner moulds, and the upper edges by the manipulation of a curved trowel. The tops of copings were brought to a smooth level finish by floating with mortar before the concrete had set.

3.—Field Engineering.

The field engineering included surveys, pier location, inspection, and general supervision of all construction.

The alignment of the viaduct was governed somewhat by property boundary lines. The structure lies in three tangents, aggregating about 6 900 ft., and three curves, each of about 1 900 ft. radius, and totaling 1 500 ft. in length.

Exact surveys and final measurements were made to locate street crossings, railways, buildings, etc., before much work was done on detailed drawings. Base lines, paralleling the chosen center line tangents were carefully marked on the ground, and the angles at their intersections determined. Points on base lines, about 400 ft. apart, were marked by large wooden hubs driven down flush. In the boggy, quaky ground these were 4 by 6 in. and from 3 to 5 ft. long, but in the firmer ground not so large. A piece of sheet copper was nailed on top of each hub, and the exact point was indicated by the intersection of two scribed lines. Each main hub was carefully referenced by marking two intersecting lines. Exact distances from hub to hub were determined either by direct measurement or by triangulation. About one-third of the entire length was triangulated. For the systems of triangles, suitable base lines, usually intersecting with the main line, were marked and measured. Measurements were made as follows: Between two main hubs, 2 by 4-in. marking stakes were solidly driven down, about 98 ft. apart; between these, at about 6-ft. centers, small supporting stakes were placed. All these were sawn off or driven down to conform to the grade from main hub to main hub. When necessary a narrow ditch was dug to contain the stakes, it

being desired to support the tape on them and not on the ground. A 100-ft. steel tape, which had been compared with a standard to determine its constants, was used. The required tension was secured with a spring balance, and the distance was marked by thrusting into a marking stake a knife blade with its sharp edge at the desired graduation. The knife was left there and measured from in placing a second knife ahead, and so on until the second main hub was reached, and there the tape was read to an approximate thousandth. Thus any number of repetitional measurements could be made without the confusion resulting from making each intermediate point with a non-erasable mark. About six measurements were made for each hub distance. The extremes varied from the mean about 1 in 80 000. The base line at the Kaw River was conveniently measured on the deck of an adjacent railway bridge, and the triangulation points there consisted merely of punch marks on the end floor-beams at fixed shoes.

The measurement of angles was done by a method of repetitions and reverses which eliminated instrumental errors in the mean of each series of readings. Enough readings were taken at each point to make the triangles close within 1.5″. The largest triangulated distance was 1 131 ft., and was determined by two triangle systems, so that the independent calculated lengths checked within $\frac{3}{32}$ in. The difference in the elevation of the two end points was 65 ft.

After all distances had been determined, the station and plus of each hub was calculated, and locations for the pedestals were measured readily from the nearest hub.

For the location of river piers, the true angular distance from each base line was calculated, and a temporary fore-sight was established by a single carefully-turned angle which was then measured exactly. The distance of the temporary fore-sight from the true line was calculated, and the point in the hub was then moved the necessary distance ($\frac{3}{4}$ to $1\frac{1}{2}$ in.). When the two reference lines and the bridge tangent were projected to the piers they intersected within a $\frac{3}{16}$-in. circle.

For pedestals, and for piers where possible, reference lines at right angles to the tangent were marked by several hubs well driven down. The longitudinal center lines of each row of pedestals were also suitably marked at intervals. Elevations along the entire line were determined, and numerous carefully checked bench-marks established.

The inspection of construction embraced every detail, and daily records of the progress of each kind of work were prepared. For instance, not only did an inspector watch the placing of concrete, but a second man watched the entire process of mixing. There was an ample number of men to make inspection of stone, timber, cement, sand, pile-driving, forms, concreting, erection, riveting, painting, paving, decking, etc., as has been noted previously.

4.—Erection of Superstructure Metal.

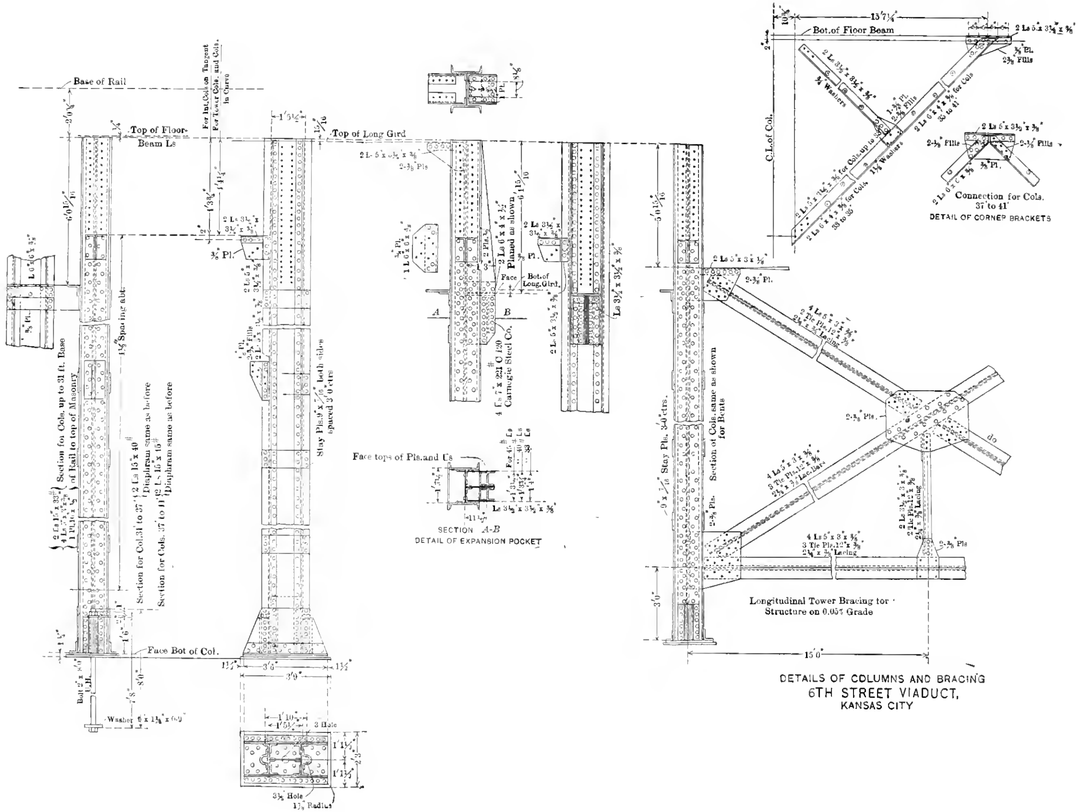
About 88% of the total length of the viaduct is of practically duplicate panels, and, in all details of construction, particularly in the erection, this characteristic feature permitted economic handling of materials and labor.

Trestle Erection.—The traveler used for the ordinary structure consisted of a pair of derricks mounted on a frame arranged to run on track rails laid on the longitudinal girders, the masts thus being 33 ft. from center to center. The details of this traveler are clearly seen on Fig. 2, Plate IV. The masts were double, one stiff and one seated to turn. Each was a 14-in. square timber 40 ft. high, and the booms were built up of spliced timbers to 14 by 26 in., and 60 ft. long. The sills and cross-sills were 8 by 16 in. All connections were made by steel plates, and steel derrick fittings were used throughout. The equipment consisted of two 2-drum, 4-spool hoisting engines of 20 h.p., each handling one boom with its lines and runners. In the ordinary structure the members varied in weight up to 10 tons, which one boom could easily handle.

A service track was laid between the pedestals nearly the entire length of the viaduct, and the steel was unloaded alongside. The intention was to distribute immediately every piece to its proper location, but, owing to the operations of the substructure contractor, this could not be done, and it was necessary to store and redistribute them. The lighter metal was unloaded by a locomotive crane, the larger girders by yard derricks, or by hand upon cribs.

It thus followed that a considerable part was erected from metal already distributed, but also a large part directly from the cars as they were run down from the yard. There was little difference in the rate of erection in the two cases, the former being retarded by the necessity of removing superposed pieces and hitching to material some-

PLATE VI.
PAPERS, AM. SOC. C. E.
FEBRUARY, 1909.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY.



DETAILS OF COLUMNS AND BRACING
6TH STREET VIADUCT,
KANSAS CITY

what indiscriminately piled, and the latter by reason of the turbulent operation of service cars over the temporary tracks. Fig. 2, Plate IV.

The pedestals were finished $\frac{1}{2}$ to $\frac{3}{4}$ in. low, and four 2 by 4-in. oak shims of suitable thickness were used to bring each column to the desired elevation. The anchor-bolts had been located so accurately that they sufficed to line up the columns, additional marks being unnecessary.

The usual order of erection per panel was as follows: Two columns were set up at once, one by each boom, and braced up by the anchor-bolts; the connecting floor-beam was placed on its shelf-angles, and bolts were put in one-third of the open holes. The cantilevers were then put up and bolted, after which the main girders were raised singly, both booms being used for the heavier girders. The stringers were then laid on top, to be distributed later, and the traveler was immediately moved forward over the new panel, ready to set the next columns. The maximum day's run was six panels, about 275 tons.

The light material in the deck was put in place in conjunction with the riveting, the stringers by using timber dollies, the bracing, etc., on a push-car running on the flanges of two adjacent stringers. However, as the traveler proceeded, it placed all sway bracing, tower bracing, and laterals.

Certain long and heavy girders required some variation from the usual order of handling. Where the columns ahead were beyond reach of the booms they were raised by a gin-pole or a gallows-frame, which then served to lift one end of the girder, the other end being lifted by the traveler. The longest girder in the structure (107 ft.), weighing 62 tons, was erected in this manner, the gallows-frame consisting of a pair of 18-in. square timbers, 60 ft. high, connected with bracing and caps, equipped with 4-sheave steel blocks reeved with steel cable. Fig. 1, Plate VII.

Because of buildings and railway tracks under the eastern seven panels, the girders could not be handled from below, but it was necessary to run the traveler back, free of obstructions, pick up a girder, set it transversely on the traveler track, move it forward by picking up, booming out, and shifting the traveler, and finally to swing it around longitudinally to its final position. The columns in the buildings were dropped down through holes cut in the roof, with small trouble to the occupants, who, however, did not object to stepping

outside for a few minutes while each girder was being manipulated above their office rooms. The girders immediately adjacent to Bluff Street extend through slots or pockets cut in the retaining wall which supports the street, and rest on pedestals 15 ft. back of, and thus entirely independent of, the wall. The tracks below these girders carry all the passenger traffic into the Union Depot, therefore unusual caution was necessary in erection to guard against possible interruption of train service.

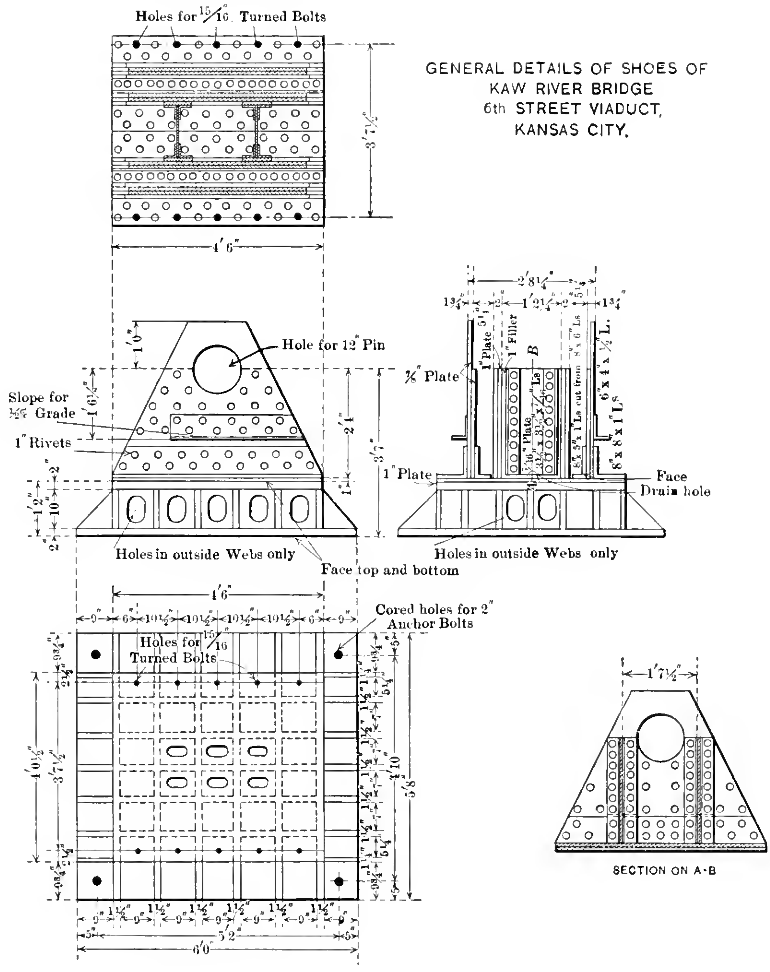
Erection of Truss Bridges.—The falsework consisted of 7-pile bents every 30 ft. under the panel points, and 5-pile bents midway between, the piles being well driven into the river bed. Four 8 by 16-in. stringers were placed under each truss, and a traveler track was supported on two runs of 12 by 12-in. timbers. This traveler was composed of two derricks rigged together, and running on a track between the trusses in the plane of the bottom chords. Beside the traveler, also between the trusses, a standard-gauge track was laid for moving out the metal.

After fastening down the fixed shoes, the bottom chords were laid on the camber blocking for the full length of the span. Beginning at one end, the web members, chords, floor-beams, and laterals, were erected in regular order.

The parts of these trusses are heavy, and the connection plates large and stiff, so that fitting together the various members was tedious work. A clearance of $\frac{1}{16}$ in. was provided, but, in a member 22 in. wide, made of four $\frac{3}{4}$-in. angles and a $\frac{3}{4}$-in. plate, which had to be entered from 5 to 6 ft. into a second member equally stiff, slight irregularities would cause binding. Practically every diagonal and chord section had to be pulled to place with a runner.

It was apparently impossible to do all the field riveting before the time of the expected washout of the falsework, and, in order to support the spans in that contingency, drift-pins were tightly driven into one-third of the open holes, and fitting-up bolts into one-third of the open holes of all main connections. Fortunately, it was not necessary to trust to this expedient, as the expected high water did not come. It is evident the spans would not have fallen, but experience showed that there would have been a slight movement at each point, which, in the aggregate, would have caused trouble.

The trusses were set carefully to correct the camber which was



FIXED SHOE FOR WEST SPAN

FIG. 5.

maintained until the erection was complete. With the passage of the time required for riveting, the falsework settled at points, permitting the entire truss to settle. Attempts were made to jack up the trusses, but the only result was a pushing down of the piles under the wedges. A regular camber of about one-half as much as estimated was finally secured. With the complete dead load and such live load as has been on the spans, the bottom chords are just about straight. The serious consequence of this settlement, however, was in the tightening of the drift-pins. These were not taken out until the rivets had been driven in all the adjacent holes, and then it was found there had been enough movement to pinch them tightly in place, requiring much energy and effort to remove them. Most of these pins had been driven toward the inside, and the cramped quarters prevented efficient striking to back them out. Many had to be drilled out, and, as they were of very hard steel, it was difficult to keep the drill point on the pins and prevent it from running off into the softer metal.

The lower chord splices of these spans were riveted first, and then the ends of the diagonals; but the upper chord splices and the intersection points of the diagonals were not reamed or riveted until the span had been swung.

The similar trusses for the 147-ft. span were erected on a falsework of framed bents set on the ground. There was no trouble in maintaining the full camber in this span; and, as the members are much smaller than for the Kaw spans, their erection was quite simple.

Riveting.—The contractor sought and received permission to ream all field holes in the field, thus making it unnecessary to assemble in the shops; and the metal as shipped out was merely sub-punched. Because of this, and since, for the unrestricted movement of various gangs, it was desirable to keep them separated, a very large amount of scaffolding was placed and retained for some time, but it was of simple character.

The joints were first thoroughly fitted up, and bolts were put in half of the holes; the other holes were then reamed, the bolts were shifted to the reamed holes, and the reaming was completed.

Each joint was thus fitted twice before the rivets were driven, but trials of reaming and then riveting half a joint, and then reaming and riveting the remaining half resulted in the interference of the various gangs, and better progress was made by the procedure given.

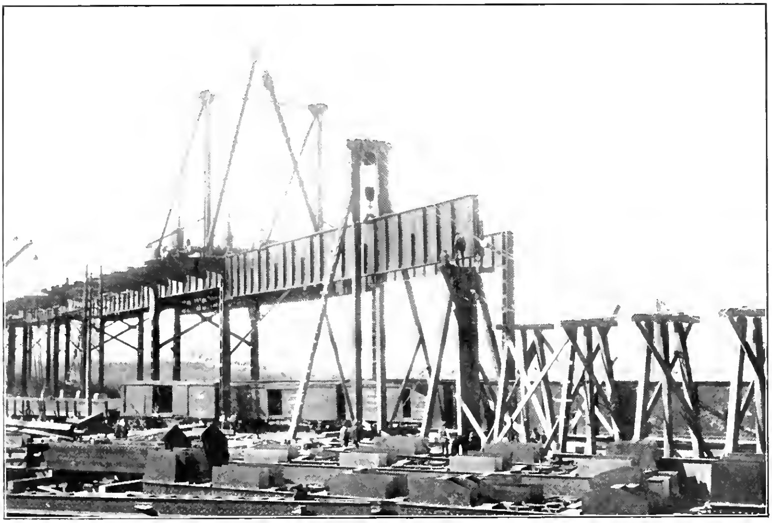


FIG. 1.—GALLOWS FRAME USED IN THE ERECTION OF LONG GIRDERS.

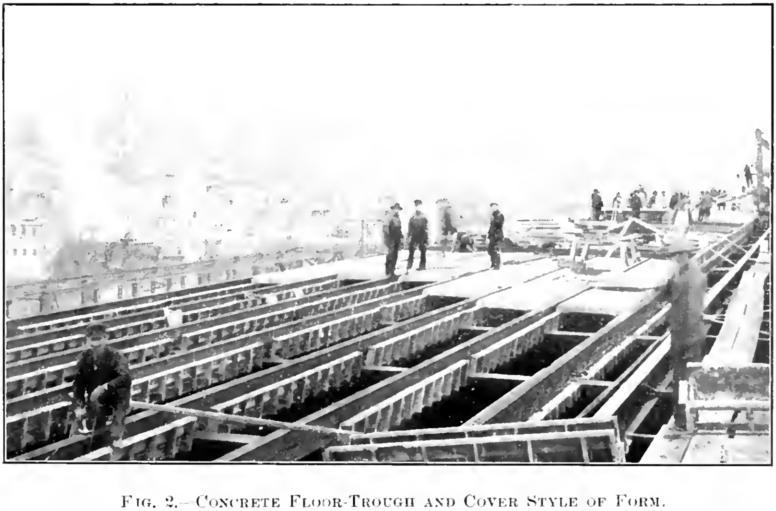


FIG. 2.—CONCRETE FLOOR-TROUGH AND COVER STYLE OF FORM.

Neglect in fitting up caused the majority of cut-out rivets, for if the plates were not drawn tightly together small particles of reamer drillings would work in between them and cause successive rivets to loosen those previously driven. Natural conditions were seized upon to assist this work; for instance, the expansion joints were regularly rammed full of steel wedges, thus gaining the thrust of expansion to force together columns and girders.

The reaming was done with Little Giant, No. 0, pneumatic drills, equipped with 4-flute, tapered reamers having barrels at least as long as the length of the hole. Water, with a little soap, was used as a lubricant, and was kept constantly flowing on the bits. A knuckle-joint was tried for points where the reamer could not be held normal to the metal, but gave indifferent results; so, when unavoidable, holes were reamed on a slight bevel, but they were required to be perpendicular to the direction of stress. There was some trouble in making all holes of the same diameter, for the reamers had to be held very steady to accomplish this. The ordinary run for two men was from 200 to 250 holes, $\frac{7}{8}$ in. in diameter, with a 3-in. grip, or equivalent, in 8 hours. The smooth, clean holes permitted the rapid driving of rivets without further drifting or fitting, and without the delay incident to entering hot rivets into imperfect holes. This was highly essential, especially in such cases as the lower chord splices of the Kaw River spans, where the rivets are 1 in. in diameter and have a 6-in. grip.

Cleveland air hammers, with a 9-in. stroke, operating under a pressure of 110 lb. were used throughout. Grip dollies were required wherever they could be utilized, and hand-forges were used almost exclusively. With one heater and two riveters, 550 rivets, of $\frac{7}{8}$-in. diameter and 3-in. grip, were driven in 8 hours.

The greater part of the riveting was open, easy work, although there were marked exceptions. The rivets in expansion pockets required special, crooked-jamb, joint dollies, in order to pass around the end stiffeners and rest on the next adjacent stiffener.

The bracing of the motorway stringers was connected by $\frac{3}{4}$-in. rivets driven by hand without reaming—in fact, it was found that the lug-angles on the beams were set so high that the rivets were under the flanges, thus precluding the use of an air hammer. Very fair results were achieved, but it would have been much better to have had wider lug-angles.

In the main structure there are about 300 000 field rivets. While accurate costs are not available, the following figures are a close approximation: Reaming cost 7 cents per hole, of which 5 cents was for labor and 2 cents for plant and tools; driving cost $8\frac{1}{2}$ cents per rivet, 8 cents for labor, and $\frac{1}{2}$ cent for plant and tools. It is possible that too much of the plant cost is charged to reaming and too little to riveting. There were reported cut-outs of $3\frac{1}{3}\%$, but apparently all the rivets were not accounted for, and 5% would be nearer the true figure.

Grouting Columns.—The space below each column base was filled with grout. A cement mortar of ordinary consistency was first prepared and a little daubed under the edges of the base plate around its entire perimeter and finished off neatly. After a day's time there would remain a tight dam all around, pierced, however, by a small hole on each side. The nuts and washers on the anchor-bolts were temporarily removed. Grout, diluted to a pouring thickness, was made in the proportion of about 1 of cement to 3 of fine sand containing a little clay. Several bucketfuls of the mixture were poured, as quickly as possible, into the anchor-bolt holes and the central hole until it ran out of all the "air holes," and then these were promptly daubed up. Mortar was added to fill entirely the side pockets; the foot of each column was filled with concrete finished off with mortar.

This method was highly satisfactory, as it gave a full and even bearing under the entire surface of the base plate, as was proven by lifting up columns which had been grouted.

For large solid castings, as under the Kaw spans, grout could not be used. The bases were set on steel shims about $\frac{3}{4}$ in. above the concrete. A stop was put at one side, and small quantities of almost dry 1:1 mortar of cement and sand were put under the opposite side, pushed clear across, and rammed with a piece of $\frac{1}{2}$-in. plate. The entire space was filled from one side, thus insuring the absence of voids.

Had holes through the casting been provided, the former method could have been used with superior dispatch and ease.

Painting.—The steel received one coat of paint in the shops, and presented a good even coating when unloaded at the site. As it was erected, all abutting parts and surfaces inaccessible after erection were coated with a heavy paint cement daubed on in generous measure.

Upon the completion of the riveting of a section of the viaduct, the entire surface of the metal was examined, carefully cleaned with wire brush and scraper where necessary, and the heads of field rivets and all areas where the shop coat was scraped off or unduly marred were touched up with paint. Reaming borings added considerably to the ordinary work of cleaning. When these spots had dried, the first field coat (of green paint) was applied. The paint was delivered ready mixed, and was of rather stiff consistency, requiring, for proper application, small amounts thoroughly rubbed on by the vigorous use of a stiff-bristle brush held almost normal to the surface.

When this coat had dried, requiring a varying time, but determined when it became impossible to "rub off" the dried surface skin and expose wet paint below, the final coat was put on. The same care was used in application, and the entire structure was finished up progressively.

The inspectors then examined the work carefully, and remained with the finishers until the painting was acceptable. Small hand-mirrors were used to reflect the sunlight and illuminate dark corners and shadowed areas, thus exposing neglected patches almost impossible to locate by any other means. In many places, too, as, for instance, inside a column, it is much more convenient to observe the reflection in a mirror than to attempt to view the parts directly.

No painting was done in wet, or wet and freezing, weather, but the work was never stopped because of low temperature, provided the surface to be coated was dry and free from sleet or ice.

5.—Viaduct Floors.

The general details of the reinforced concrete floor are shown on Fig. 3. The average slab thickness is 6 in. above the stringers, the stringers being entirely encased, both to protect them from the smoke of locomotives below and to secure the added strength of the filled haunches. The amount of reinforcement is considerably in excess of actual load requirements, in order to provide for the temperature stresses of the concrete. It consists of transverse and longitudinal bars, near both the upper and lower surfaces of the slab, of extra bars over the cross-beams, of numerous vertical bars in the curbs, and of wire netting around each I-beam stringer. To support the hand-rail

posts, bolts project through the sides, the lower bolts passing through the I-beams.

In order that the usual paving could be continued without interruption over the points of expansion, a flexible connection was devised consisting of U-shaped plates bolted side by side. Thus a sort of bellows trough is formed, about 6 in. high and 2 ft. wide. One such bellows extends across the roadway at each expansion joint of the stringers, and is riveted to each stringer on both sides of the opening. The plates are thin enough to spring easily under the movement of the steel. The concrete is finished up to each side of the bellows, and fastened to it by bolts. These troughs were to be filled with a rather soft asphaltic material, the flow of which would accommodate it to the varying position of the containing plates.

The specifications provide for concrete made of cement to be supplied by the company; of clean, hard, broken stone "free from all dust, dirt, shale, rotten stone, or other objectionable material", to pass a 1-in. ring; and of Kaw River sand in the proportions 1 : 3 : 5 by volume, loose. To provide a bond for the contemplated wearing surface of asphaltum (which was to be laid without the usual binder course) it was required that clean stones be sprinkled on the surface of the wet concrete, and be tamped into it enough to embed them partially, yet permit them to project a little. Other principal specifications were: That the entire thickness of the floor was to be deposited in one continuous operation so as to avoid laminations or dry horizontal planes between consecutive layers; that all surfaces were to be finished neatly to the exact required dimensions; that all bolts and embedded castings were to be set to exact position; that reinforcing bars were to be spaced accurately; and, finally, that the contractor was to clean all stains and concrete droppings from the metal work.

The motorway stringers at the side of the roadway were used as a runway for the concrete mixing equipment. A standard-gauge track was laid on these stringers for some distance, to accommodate the contractor's plant, which he placed on several railway flat cars. This plant consisted of two large elevated hoppers, connected by a cable carrying a conveyor bucket, and a belt conveyor for handling finished material, and similar appliances. Much time was consumed in the arrangement of this equipment which was soon found to be unsuitable. The company's engineers then designed a new and simple plant, the

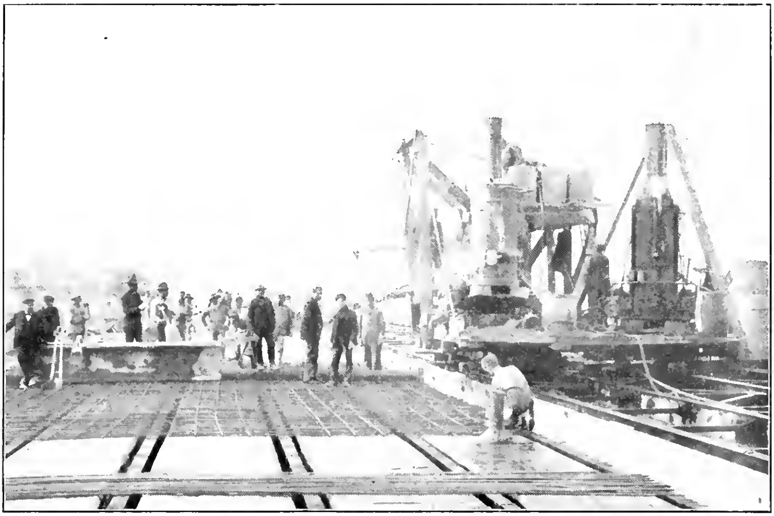


FIG. 1.—ARRANGEMENT OF CONCRETE PLANT FOR LAYING FLOOR

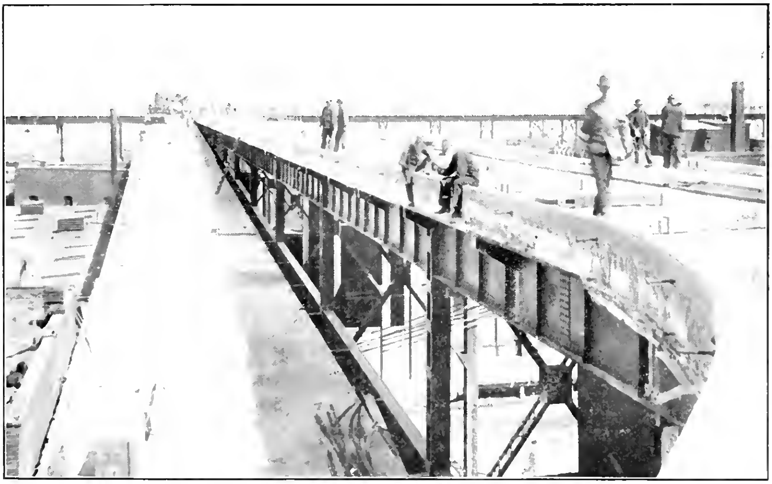


FIG. 2.—FINISHED FLOOR OF MULBERRY AND HICKORY STREET APPROACHES.

old elaborate one was dismantled, and five of the smaller plants were quickly built and put into successful operation.

The plant is shown in Fig. 1, Plate VIII; it consisted merely of a platform of timbers, mounted on wheels to run on the motorway stringers, supporting a mixer, a derrick, and an engine. The derrick raised the aggregates from below with a skip box, and a metal-lined chute extending over the roadway received the concrete from the mixer. The total cost of one plant, including all fittings except the mixer and the engine, was $700.

Forms.—Several arrangements of forms were tried, and, as these made up a considerable part of the floor concrete cost, they merit special attention.

The contractor first furnished all-metal forms, made of corrugated sheets pressed to shape and covered with light sheet iron. They were made in sections 5 ft. wide and about 5 ft. long, the longitudinal junction of various sections being immediately under the stringers. Along each of these lower edges there was fastened a small angle having holes in its vertical leg. Keys were provided to pass through these slots to fasten adjoining forms together laterally. Provision was made for the forms to lap about 1 in. longitudinally, but there was no way of fastening them together. To support these forms, three joists were placed from cross-beam to cross-beam, and three sections of form were laid thereon. Numerous imperfections were soon evident. The form had almost no lateral stiffness, each unit having an individual shape. At best, the surface presented was not true; the correct distance was not left on each side of and below the I-beams; there were openings and cracks at the transverse joints, as these could not be made to lap as intended; many pieces had to be held during adjustment; and, also, the forms were easily displaced by the ordinary walking about and working on them.

Owing to the non-delivery of the metal forms, the contractor had already begun to build wooden forms of much the same type, that is, of sections to meet under the stringers and of such length that three were put between adjacent cross-girders. They were built of 1 by 4-in. frames spaced 18 in. from center to center, sheeted with ½-in. boards, and covered with No. 14 sheet iron. These forms were hung on the stringers with two ¼-in. round, soft iron rods on each side. After adjusting the form to position, the hangers were bent around the upper

flanges of the **I**-beams. To remove the forms, the hanger rods were cut with heavy clippers close to the sheeting. These wooden forms were light and stiff, but they, too, showed objectionable defects. There were no means of really fastening the various units together, and there were presented, not only the transverse cracks of the metal forms but also a longitudinal crack under each stringer, and these latter were almost inaccessible for inspection or adjustment. Nothing was found to prevent water and cement from leaking out.

The company's engineers then designed a form, and the use of the others was discontinued as soon as they could be replaced. The finally acceptable form consisted of two parts: **U**-shaped troughs which were placed under the stringers; and covers which extended from trough to trough between stringers. They were of wood, 1 by 4-in. frames, with $\frac{1}{2}$-in. or $\frac{3}{8}$-in. sheeting, and covered with No. 16 or No. 18 sheet iron. The troughs were 13 ft. 10 in. long, or just enough to go easily between the cross-beams; the covers were of various convenient lengths, from 4 to 6 ft.; and the outside curb faces were 18 ft. long. To support the troughs, $\frac{1}{4}$-in. round iron hangers were used, a loop-shaped piece being put around the 2 by 4-in. sill on the bottom, and each leg being bent over the upper flange of the **I**-beam. Four or five ties were used for each section. To hold this trough in correct position and prevent rotation, briquette spacers were used. These were made to fit the lower curve of the form, and had a shoulder on each side at the top to fit the lower flange of the **I**-beam.

A nest of wooden moulds was provided, and briquettes of sand and cement were made in advance of the need, so as to be thoroughly hardened. One briquette was placed at each end of each trough (thus closing it against leakage) and one under each hanger. When the hangers had been drawn reasonably tight and bent over, the trough was so rigid that a man could walk along one edge.

The cover forms, it was found, could be handled with a lap at one end, and, when the troughs were in position, could be rapidly laid, producing a continuous unbroken surface. The small remaining vertical hole (8 by 12 in.) at each floor-beam was covered with a piece of No. 8 sheet iron. The forms for the outside faces of the curbs were made of $\frac{7}{8}$-in. plank. These were stood on the already erected curb trough, adjusted to position, and nailed lightly. A horizontal hanger held them laterally, and they could be made truly vertical.

These forms proved to be highly satisfactory, presenting smooth, rigid, correctly-shaped surfaces, with very few chances of leakage.

They were easily altered to meet conditions, and stood rough treatment well. Their erection and removal was simple, any part being handled by two men without extra scaffolding. Fig. 2, Plate VII.

The briquette detail was one of the most advantageous features, and was developed by a prior use of wooden blocks, which gave trouble by breaking out the concrete as they were removed with the form. The cement briquettes remained tightly in place, and panel after panel of floor was built without a place to touch up below the beams.

The cost of building 600 lin. ft. of these forms for the 30-ft. roadway was:

Lumber	$1 160
Sheet iron	650
Nails and sundries	165
Labor	1 300
Contractor's profit	325
	$3 600

The first cost was $6 per lin. ft., or $0.20 per sq. ft. between curbs. Forms used five times cost 4 cents per sq. ft., or $1.30 per cu. yd. Many of these forms were used oftener than five times. The fact that they were built in great haste doubtless increased their cost somewhat.

Making and Placing Concrete.—Owing to difficulty in securing a sufficient quantity of 1-in. stone from local crushers, the use of flint screenings from zinc mines was permitted. Although this material is clean, hard, and sharp, and is somewhat graded in size, needing but little sand, there are objections to it. A few moments after it had been deposited quite wet, the water appeared to drain away, leaving the upper stones clean and dry, without any mortar coating whatsoever, and the entire mass appeared to be inert and "dead". This appearance was evident even while the concrete was still green enough to be dug through with a stick. After several days the main body became hard and compact, but on top there was perhaps ¼ in. of loose stones. Various mixtures were tried, to see if the concrete could be made to remain "quaky" for a time, but it could not be done. Yet experiments on the

tensile strength gave creditable results, and investigations by cutting through slabs thoroughly set showed the concrete to be very hard. About half the floor concrete was made of this flint, and the remainder of crushed limestone. The latter, of course, could be flushed with mortar, and gave normal results.

All concrete materials were delivered on the ground under the viaduct. Skip boxes were filled with a batch of correctly proportioned aggregates and raised to the mixer above. The concrete was dumped into a trough and slid down to be distributed over the floor. To expedite the filling of the small space below the I-beams, cement mortar was used to a height of about 2 in. above the lower flanges.

The reinforcing bars were spaced and held in position as follows: Round $\frac{3}{4}$-in. iron rods were laid longitudinally on the forms, about 5 ft. apart, and on these the lower transverse bars were placed. The lower longitudinal bars were then laid in position and fastened at numerous intersections with wire ties. When some 3 in. of concrete had been spread, a rope was fastened to the ends of the $\frac{3}{4}$-in. rods and a strong pull slipped them out longitudinally from the finished work. A very minute inspection of the whole floor failed to show a single instance of a lower bar exposed.

Combined supports for the upper bars and guides for finishing the upper surface consisted of 2-in. joists having small angles fastened on each side and a series of hooks in the lower edge. They were placed longitudinally with the structure and 10 ft. apart, and were held on small blocks resting on the stringers. The hooks supported the transverse upper bars in correct position. A gauge, sliding along the angles, gave the desired surface shape to the concrete.

The concrete was first tamped with a large flat tool and, after hardening somewhat, was retamped with a corrugated tool. The latter was made of wood with parallel cleats, $\frac{3}{4}$ in. wide, $1\frac{1}{2}$ in. deep, spaced 2 in. apart. The cleats were sharpened slightly. With proper manipulation this produced light corrugations, in varying directions, of $\frac{3}{16}$ to $\frac{1}{4}$ in. variation, thus giving a satisfactory bond for the asphaltum. Scattering loose stone and tamping it in was tried, but the former roughening was superior. Neither method gave entire satisfaction where flint was used.

The plants were moved forward as each section of about 15 ft. of roadway was completed. On an average, the five plants placed from

300 to 350 lin. ft. per day; the best day's run for one gang was 98 lin. ft., or 90 cu. yd.

The floor was finished level to the edges, and the curbs were built later, in the usual way. Bars projected upward into this curb every 12 in. and bound it to the body of the work.

With all types of forms, and to a pronounced degree with the first designs, there was considerable leakage from the wet concrete. This dripping water carried such a quantity of cement as to leave a whitish deposit on the metal where it drained, and in certain cases spattered down on the ground under the structure. Probably the actual loss of cement was small, but although it was easy to know how much cement went into each yard, or each day's run, no scheme was discovered to ascertain how much actually remained in the concrete. This cement was not cleaned from the metal promptly, and the cleansing later proved troublesome. The plan was to wash the metal with a strong stream of water immediately after the concrete was placed and until it ceased to drip. Adequate pumps and lines were not installed to do this, and finally much cleaning had to be done with wire brushes and scrapers, necessitating some repainting.

The entire concrete floor was built in 67 working days, during which there was some freezing weather. The contract provided for heating any or all concrete materials, but none was heated other than some sand and this merely to thaw frozen lumps. However, wagon sheets were spread over the green concrete each night, and, when hard weather was expected, these were covered with stable sweepings. No bad effects were found on the floor proper, but some portions of curbing were frozen and had to be torn out and replaced. Some of the concrete was laid when the temperature was 31°, and the following days were about the same, yet in 10 days' time asphalt rollers were run over the work.

The 30-ft. roadway contains 24.6 cu. ft. per lin. ft. and the 38-ft. roadway 33 cu. ft. per lin. ft., or an average for both of 0.84 cu. ft. per sq. ft. of area between curbs. The average amount of reinforcing is 2.67 lb. per sq. ft.

The main viaduct contains about 8 000 cu. yd. of concrete. The contract cost, exclusive of cement and bars, was $6.085 per cu. yd. The cost of completed floor, not including paving, was about 36⅓ cents per sq. ft.

Paving.—The paving is of Bermudez asphalt. The following results are from average analyses:

Specific gravity	1.098	
Insolubles	2.0%	
Soluble in 88° naphtha	63.0	
Soluble in 62° naphtha	85.0	
Penetration at 77° Fahr.	38°	(Dow.)
Fixed carbon on ignition	14.5%	
Ductility	20.	(C. M.)

This hard asphalt was softened by the addition of asphaltic petroleum residuum which tested as follows:

Specific gravity, Baumé	11.25
Boils	83° cent.
Flashes	173.5° cent.

From 18 to 20 lb. of this oil were mixed with 100 lb. of asphalt. The hardness of the resulting mixture at 77° Fahr. varied from 67 to 78° (Dow).

It is rather difficult to secure just the sand desired for asphalt in Kansas City, as the Missouri River sand is somewhat water-worn and the finer Kaw sands contain some loam. Pulverized limestone and cement were added to the sand to give the required 200-mesh material.

The following mix was taken as a criterion, although the specifications gave somewhat more latitude:

Bitumen	9.5%	to	11.5%
Sand passing 200	6 %	"	16 %
" " 100	8 %	"	16 %
" " 80	10 %	"	16 %
" " 50	14 %	"	34 %
" 40	9 %	"	16 %
" " 30	6 %	"	10 %
" " 20	3 %	"	7 %
" " 10	2 %	"	4 %

The paving material was given close inspection, both at the plant and on the work. Samples, taken daily, were analyzed and sifted, and daily directions were given to the mill. On the viaduct, samples

were taken from 1 in every 5 to 15 loads, depending on the rapidity of delivery, etc.

The rolling was done carefully with 6 and 8-ton rollers. Where necessary, the surface was ironed, with especial attention to securing adequate drainage. The entire pavement was placed in 20 days.

The iron trough expansion joints were filled with the ordinary mixture. At some of these the wearing surface cracked during cold weather, and in the later work thin separators were inserted to make these openings straight across the roadway. These joints are acting as expected, and are considered very satisfactory.

Asphalt was not laid on the new embankments, but a Macadam pavement on a very heavy base was substituted and finished with oil.

The contract cost for the asphaltum was $1.05 per sq. yd., and for the Macadam, $1 per sq. yd. Thus the concrete floor with pavement cost about 48 cents per sq. ft.

Street-Car Deck.—The ties of the street-car deck are of creosoted, short-leaf, Southern pine, 6 by 8 in., laid flat, and spaced 13 in. from center to center. Every fourth tie extends across both tracks, thus serving to brace the whole deck together and support a possible runway. Guard timbers are dapped 1½ in. on the ties and are set 11 in. beyond the gauge. On curves, the ties are on edge, the variation in the depth of the daps giving the outer-rail elevation.

All the timber was surfaced on four sides, and finished to full size. The general creosoting specifications were the same as already given for piles, from 10 to 13 lb. of pure dead oil of coal-tar being used per cubic foot of timber. The dapping, boring, and cutting were done before creosoting. The deck timber cost delivered $45 per 1 000 ft., B. M.

A small derrick car, equipped with a gasoline hoist running on the stringers, raised the material to the decks from its distributed position below. The ties were set to approximate place and the guard rails laid. Wooden rams were used to drive the guard rails down on the ties, to shift the ties to fit the guard-rail daps, and to settle them on the stringers. Then the hook-bolts and guard-rail bolts were inserted, and the deck was tightened to place with socket wrenches on auger handles.

The rails are of 70-lb., Am. Soc. C. E. section, 30 and 33 ft. long, laid with broken joints, fastened to every tie with two ordinary 5½-in. spikes. Six-hole angle-bars were used.

The tracks are bonded by two copper bonds at each joint, and set up with a screw compressor. For each bond, two $\frac{7}{8}$-in. holes were drilled through the rail web. Some of these holes were drilled with hand-ratchets, but as the rails were very hard and the drilling was slow, a machine drill was substituted. This consisted of a $1\frac{1}{2}$-h.p. gasoline engine, set on a push-car, operating a flexible shaft, which was attached to an automatic-feed drill working in a yoke that fitted over the rail. This machine cost only $100, in addition to the engine, and gave creditable results. It would drill the four holes at a joint and move forward in 12 min. A very weak solution of sal soda was used for lubrication.

The cost of laying deck timber was about $5 per 1 000 ft., B. M.; for laying rails about 4 cents per lin. ft. of single track; the work of bonding cost about 50 cents per joint.

The usual overhead trolley is used. The poles are of iron, from $6\frac{1}{2}$ to 5 in. in diameter and 20 ft. high above the ties. They are placed between the two tracks, about 90 ft. apart, and support cross-brackets for the wires.

Hand-Rail.—The hand-rail was designed for strength, simplicity, and architectural harmony with the structure. It consists of square, cast-iron posts, 9 ft. apart, supporting horizontal rails of ordinary pipe, the diameter of the upper one being 3 in. and the other four $2\frac{1}{2}$ in. Each post is fastened by two bolts embedded in the concrete floor, the upper of which has two nuts, in order to provide for the adjustment of the post. The pipe rails pass through holes in the posts, fitting rather loosely, and are joined by standard unions and made continuous. At each post, and for each pipe, two small steel wedges are driven into seats, gripping and pinching the pipe and holding it tightly in place.

Thus the hand-rail could not be erected until the floor was finished, and then the posts and rails had to be put up simultaneously. A small two-wheeled truck was used to set up the posts. The forward end of this truck had a cantilevered fork on which a post was supported as it was drawn on to the projecting bolts. The railing pipes were inserted consecutively and jointed together as the successive posts were erected.

When several hundred feet of railing was in place the posts were adjusted to points set by a transit, thus bringing the upper rail to a true line. The cost of the hand-rail is included in the general schedule price for superstructure metal erected.

Lighting System.—The annoyances of a trip through the bottoms are aggravated by darkness, and an adequate lighting system for the accommodation of night travel was considered highly desirable for the viaduct. Natural gas is available, and was considered, as also were other plans for lighting, but an incandescent electric system was selected. The results are exceedingly satisfactory; the illumination is uniform and not glaring, yet so bright that a newspaper can be read anywhere on the roadway.

Every fourth hand-rail post is extended above those intermediate, and supports a 16-c.p. Tantalum lamp, set in a vertical, moulded, mica socket, surrounded by a glass globe 10 in. in diameter. There is thus a light every 36 ft. along each hand-rail. The lights are uniformly 8 ft. 6 in. above the pavement, and are staggered on the two sides, making one lamp for each 18 lin. ft. of roadway.

The wiring is of the three-wire, 250 to 500-volt system, with two lamps in series, connected in three circuits so that any third of the lamps may be used without changing the factors of symmetry and distribution. Certain main posts have lamps wired on the three circuits so as to show light regardless of which third of the system is in use.

In a central office is installed a switch-board, a motor generator, and certain other machinery. From this office are run two sets of three circuits, each with a common neutral for each set, to furnish current for the lamps of half of the structure, and two similar sets of three circuits each to furnish current for the lamps of the other half. The two outside wires of each circuit, together with its series wires, are run in one metal conduit, and the neutral for each set of three circuits is run on glass insulators.

The conduits and wires are supported on short wooden cross-arms fastened on the top of each floor-beam between the two outermost stringers on each side. They are thus well sheltered, on each side and above, by the concrete floor.

From the office to the structure the four neutrals are combined into two wires, each of solid copper, No. 10, B. & S. gauge. Elsewhere, each neutral consists of a solid copper wire, No. 12, B. & S. gauge. The wires connecting two lamps in series are of the same character, and of No. 14 gauge. Where placed on glass insulators, the neutral wire is of single-braid, Okonite insulation, and all feeders and branches run in metal conduit have double-braid insulation. Joints in the wires

are soldered and double-taped with Okonite and friction tapes. The conduits are of loricated metal, and where branches are taken off, Crouse-Hinds condulets are inserted.

The feeder wires of the lamp are led down through the post to a fuse-box fastened near its bottom, in which is a 6-ampere cartridge fuse. To reach the fuse-boxes from below the floor, a 2-in. pipe was placed in the concrete at each lamp-post, and through this is passed a light $\frac{3}{4}$-in. metal conduit, bent to prevent the entrance of water, and bushed at each end.

All branch wires running under feeders from the conduit are encased in continuous lengths of Alphaduct. Corresponding with the variation of the size of the wires, the conduits are varied, the six carrying main feeders being $1\frac{1}{4}$, 1, and $\frac{3}{4}$ in. in diameter, respectively.

There was serious trouble in entering the several wires into the conduits, because of their comparatively small size; slightly larger conduits would have simplified the task.

The company furnished the lamps, globes, and certain castings, but the contractor installed them, and furnished and installed all other equipment for $28 000. There are about 500 lights on the main viaduct.

6.—Approaches and Laterals.

As has been stated, legal complications prevented the main lateral on Mulberry Street from being built as at first planned, and it became impossible to effect a landing at 12th and Liberty Streets. This lateral was always considered of vital importance to the success of the enterprise as a whole, and when the original plans had to be modified it was decided to build to 12th Street—two blocks east of the former corner—and there install electric elevators of sufficient size and capacity to raise to the deck of the structure any vehicle with its horses. Further delays of a legal nature arose, however, and at present, December 1st, 1908, the Mulberry Street lateral is built to the corner of Mulberry and 9th Streets, and two elevators are installed there. The Hickory Street branch of this lateral divides from the main branch about 300 ft. from the main viaduct, runs down on a 3% grade, turns under the former, crosses Mulberry Street, and ends at 9th and Hickory Streets. This branch is shown on Fig. 2, Plate VIII.

These laterals provide only for the highway loading, the Mulberry Street portion having a roadway 24 ft. wide, and the Hickory Street

branch a roadway 20 ft. wide. The pavement floor, hand-rail, and details are the same as for the main viaduct. The detailing of the steelwork is very similar, but of very much lighter construction; it is a two-column structure with longitudinal girders, transverse beams, and longitudinal stringers. No towers or longitudinal bracing are used; the columns are fastened down with four anchor-bolts which provide the necessary stiffness in every direction.

The foundations are principally of small concrete pedestals resting on sand. They are placed at a sufficient depth below the street grade to allow for cellar construction. Two of the elevators have been installed. Each has a counter-weighted moving platform, 11 by 32 ft., lifting a total distance of 40 ft. 9 in. They are designed to carry 22 000 lb. at 50 ft. per min., or 11 000 lb. at 100 ft. per min. One 50-h.p. motor is used for each elevator. Automatic gates are provided above and below and on the platforms.

These two approaches were finished and opened to traffic a few months after the completion of the main structure.

The distances saved by the viaduct are shown by the following computation:

Distance between Eighth and Wyandotte Streets and Fifth Street and Minnesota Avenue.

By Metropolitan Street Railway line..........	11 950 ft.
By Sixth Street Viaduct....................	8 400 "
Saving.............................	3 550 ft.

Distance between Eighth and Wyandotte Streets and Fifth Street and Minnesota Avenue.

By Metropolitan Street Railway line..........	16 000 ft.
By Sixth Street Viaduct....................	11 850 "
Saving	4 150 ft.

7.—Conclusion.

The entire construction of the main viaduct was completed in 18 months; in fact, the work was done with such dispatch that several contracts were finished ahead of their allotted times. The substructure was ready for steel in 7 months. The steel erection occupied 10

months, the concrete floor was in construction 3 months, and the asphalt was laid in 1 month.

It is not possible, without lengthy explanatory detail, to give exact distributed costs of the complete structure. These figures have been compiled and compared minutely with the preliminary estimates. When due allowance is made for various changes, additions, and deductions, which originated after the first estimates were prepared, a very close agreement is seen. The greatest variation of actual cost to estimated cost is to be found in the items "right of way" and "legal expenses", both of which are materially in excess of the estimates.

Contracts were made on the basis of unit-price payments for quantities, and provision was made for all anticipated contingencies. The extras on all contracts were performed at actual cost plus 10 per cent. The total amount of such bills on the two principal contracts was about $\frac{8}{10}$ of 1% of the total final estimates.

The total amounts paid to the contractors, and the total cost to the company, of the principal physical features of the structure are given below, with the unit-prices for the work.

The costs refer to the Main Viaduct, which includes the Kaw River Bridge.

Substructure:		
All pedestals, abutments, earth work, etc.	$283 000	
Three piers in the Kaw River	98 000	
		$381 000
Superstructure:		
Steel erected and painted	$785 000	
Concrete floor	95 000	
Pavement	35 000	
Street-car deck and trolley	86 000	
Lighting system	28 000	
		1 029 000
Right of way		227 000
Total		$1 637 000

Unit-Cost of Work.—

Concrete in pedestals, including excavation, forms, etc. ....$10 per cu. yd.
Concrete in shafts of river piers..... 11 " " "

Mass of foundations of pneumatic piers. (This comprises the gross volume of base, including timber, iron, concrete, and cost of sinking)	$18 per cu. yd.
Anchorage metal in place	$4\frac{3}{4}$ cents per lb.
Creosoted piles furnished at site	46.40 cents per lin. ft.
Driving creosoted piles, measured below cut-off	30.25 to 31.35 cents per lin. ft.
Concrete piles in place	88 cents per lin. ft.
Structural metal delivered, erected and painted	3.31 cents per lb.
Construction of concrete floor (cement and bars furnished)	$6.085 per cu. yd.
Asphalt pavement	1.05 per sq. yd.
Laying ties for street-car deck (all material furnished)	5.00 per 1 000 ft., B. M.
Laying, bolting, and spiking rails	4 cents per lin. ft. of single track.

Costs per Linear Foot.—

Viaduct with 38-ft. Roadway, and Two Car Tracks.

Substructure	$38.15 per lin. ft.
Steel	92.60 " "
Floor and pavement	18.95 " "
Street-car deck	10.75 " "
Lighting system	3.50 " "
Total cost	$163.95 per lin. ft.

Viaduct with 30-ft. Roadway, and Two Car Tracks.

Substructure	$38.15 per lin. ft.
Steel	88.75 " "
Floor and pavement	15.35 " "
Street-car deck	10.75 " "
Lighting system	3.50 " "
Total cost	$156.50 per lin. ft.

Kaw River Bridge, 30-ft. Roadway and Two Car Tracks.

Substructure	$163.00	per lin. ft.
Steel	198.50	" "
Floor and pavement	15.35	" "
Street-car deck	10.75	" "
Lighting system	3.50	" "
Total cost	$391.10	per lin. ft.

The small difference in cost between the 30-ft. and the 38-ft. roadways is accounted for by the fact that the former is built strong enough in all details to allow for widening it to 38 ft. by lengthening the roadway cantilevers.

Considering both the roadway and the total width of both street-car tracks as upper surface area, the costs per square foot of this area were:

For 38-ft. roadway portion	$2.66	per sq. ft.
For 30-ft. roadway portion	2.93	" "
Average for viaduct proper	2.835	" "
For Kaw River Bridge	7.32	" "

Personnel.—This viaduct has been a work of many minds and many hands, and any list of those associated with it must necessarily be incomplete. The structure was designed by, and constructed under the supervision of, the firm of Waddell and Hedrick, composed of J. A. L. Waddell, and Ira G. Hedrick, Members, Am. Soc. C. E., Consulting Engineers. (This firm has since dissolved.)

Mr. J. H. Thompson, of New York, was Chief Engineer, and had, as his local representative, Mr. E. S. Whitney. V. H. Cochrane, Assoc. M. Am. Soc. C. E., was Engineer in Charge of Shop Inspection. The writer served as Resident Engineer for the Consulting Engineers, and was in general charge of the construction.

The general contract for the substructure was in the hands of Mr. James F. Halpin of Kansas City, under whom were various subcontractors. The pneumatic piers were built by Kahmann and McMurray, of Kansas City. The concrete piles and most of the timber piles were driven by The Foundation Company, of New York, under the personal direction of the late George Adgate, M. Am. Soc. C. E. The steel was furnished and erected by the Riter-Conley Manufactur-

ing Company, of Pittsburg, with Mr. John P. Wagner as local representative.

The concrete floor was built by The Expanded Metal and Corrugated Bar Company, of St. Louis, under George R. Heckle, Assoc. M. Am. Soc. C. E. The asphalt paving was laid by the Parker Washington Paving Company, of Kansas City. The laying of the street-car deck, certain substructure, and other various work was done by H. C. Lindsly and Son, of Kansas City. The lighting system was designed by Messrs. Weeks and Kendall, Consulting Engineers, and installed by The Squire Electric Company.

The writer remembers with appreciation the efforts of a number of assistant engineers and inspectors, who labored zealously and efficiently. Grateful thanks are due to many gentlemen for aid in collecting the foregoing data, particularly to Messrs. Waddell, Hedrick, and Cochrane. L. R. Ash, Assoc. M. Am. Soc. C. E., was identified throughout with the designing and preparation of the plans.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

SAMPITTIC SURFACING.*

By Walter Wilson Crosby, M. Am. Soc. C. E.

To be Presented April 21st, 1909.

For several years the writer has been studying the uses (and results) of hydrocarbon compounds in road making, and has been impressed with the lack of standards by which to judge the various so-called "bituminous cements" offered for sale, as well as with the pressing need for such standardization. With the increased use of this cement has come the inevitable increase in the variety of trade compounds; moreover, with the change in traffic conditions, highway engineers tend to conclude that the need for such a proper cement, and its future use, are greatly increased.

The writer, from personal inspection, is familiar with most instances of the use of hydrocarbon cements in road building in the Eastern States, and thinks it will not be disputed that cements of considerable variety have been used and many differing results have been secured.

Under different conditions, occasionally, similar results have occurred; and, under similar conditions, varying effects have been obtained. Analyses of the causes for these variations seem to indicate that differences in the hydrocarbon compounds used are the main

* Gr., psammos, sand; pitta, pitch.

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sources of variation, and that, if proper standardization of the cement material could be secured, one might hope for uniformity of results. To this end the writer, for more than a year, has been conducting a search for information concerning the composition, from the standpoint of the highway engineer, of the hydrocarbon compounds available, and the effects, when used on the road, of certain constituents known to exist. He regrets to report a woeful lack of published information on the subject, this, apparently, being due to the complexity of the compounds dealt with, and the unscientific way in which many who are most familiar with the use of the material in question have made and recorded their experiments. The results attained by the writer are far from perfect, on account of the limited time at his disposal for this work, and, quite likely, his limited abilities in this direction. He hopes, however, that by directing attention anew to the subject, and possibly by furnishing a concrete starting point for further work, his efforts will not have been in vain.

It will be recalled that, not so many years ago, it was the custom to specify cement of a particular brand, or cement which, in the opinion of the engineer, was equal to the product of a certain named firm. To-day lime cement is a standardized article, and such specifications would be looked upon with contempt.

Is it more than fair to hope that the standardization of bituminous cements may follow shortly; and to believe that, not only is it possible that within a few years the engineering profession will be relieved of criticism for such specifications as "Borrowin's Pilgrim Brand Joint Filler," "Rotto Asphalt," "Stickulithic Processes," or "equally good" thereto?

The specifications for "Sampittic Surfacing," offered herewith, may be used as a starting point for discussion, principally because it seems to the writer that the simplest form of the use of a hydrocarbon cement is thus presented.

Further, it seems possible that, if a proper standardizing of a base is secured, modifications or solutions of this base, required under different conditions and for different purposes, may easily be made by simple changes in the inserted figures, as the necessity may dictate.

The writer will gladly welcome discussion of the foregoing, and of the specifications appended, especially those clauses relating to the "Pitch Compound."

Specifications.

Sampittic Surfacing.

281.—Sampittic surfacing shall be used wherever provided for in the plans or directed by the Engineer. The width and thickness required at different points shall be those shown on the plans, or designated by the Engineer.

282.—This sampittic surfacing will usually consist of one 6-in. course of a thorough mixture of pitch and sandy soil, to be formed on the roadbed after the latter has been properly graded and shaped.

Roadbed Material.

283.—The roadbed for the sampittic surfacing shall consist of the natural sandy roadbed prepared and shaped as shown on the plans. Where the natural roadbed is found, during the work, to consist of material which, in the opinion of the Engineer, is unfit for this form of construction, the Contractor shall remove as much of this unfit material as may be deemed necessary by the Engineer and shall replace it with sandy material which, in the opinion of the Engineer, is suitable for the purpose. All unfit material thus removed will be measured and paid for as "Excavation."

Cuts and Fills.

284.—In cuts and fills, unless otherwise specially directed, the roadbed shall be graded to a width of —— feet, and shall be free from spongy and vegetable matter, roots, and stumps. The portion of the roadbed prepared for the pitch surfacing shall be —— feet wide, and shall be brought to the grades and cross-sections shown on the plans. The fills shall be —— feet wide on top.

Old Earth Roadbed.

285.—Where no change from the present grade of those portions of the road, not already surfaced, is shown on the profile, the roadbed shall be shaped to the proper cross-section; and slight elevations, with contiguous depressions, shall be removed, so as to form an even and smooth surface.

Shape.

286.—The shape of the finished road shall be that shown on the accompanying plans, and shall have a cross-slope of —— inches to one foot.

Pitch Compound.

287.—The pitch compound to be used shall be a hydrocarbon, refined from tar or petroleum, or made from natural asphaltic pitches, by the use of a proper solvent or flux.

It shall be uniform in character, and shall possess the following characteristics:

a. The percentage of free carbon shall not exceed ten.

b. The percentage of fixed carbon shall not exceed twenty.

c. The percentage of paraffin shall not exceed two.

d. When 20 grammes of the material are heated on a flat-bottomed dish, 2¼ in. in diameter, for 24 hours, in an oven, the interior of which is maintained at a uniform and constant temperature of 105° cent., the loss shall not be more than 15 per cent.

e. A sample of the pitch compound proposed to be furnished will be evaporated until it has been hardened to show a penetration, by the Dow machine, of 50 cm. at 25° cent. This pitch must show a ductility, by the Dow machine, greater than 50 cm. at 25° cent., and greater than 10 cm. at 10° cent.

f. When an amount equal to one-half the percentage of loss, under the requirements of Clause *d*, shall have been distilled from the normal pitch compound, the residue shall show a consistency of not more than 40 cm. at 35° cent.

g. The consistency of the pitch compound for this work shall be such that 50 cu. cm., at a temperature of 100° cent., will flow through an Engler instrument in not more than 75 sec.

h. When the pitch compound proposed to be used shall have been evaporated to a consistency which will show a penetration of 50 cm. at 25° cent., the percentage of this residue to the whole sample of normal pitch compound shall be not less than 60.

i. The melting point of this residue shall be not less than 65° nor more than 85° cent.

j. The pitch compound proposed to be used shall contain no constituents which can be removed by distilling it up to 170° cent., nor more than 20% by distillation between 170° and 270° cent.

k. All pitch compounds must be free from dirt or adventitious matter in excess of 2% of the compound in its normal state.

l. Pitch compounds which, in the opinion of the Engineer, have

offensive characteristics, by reason of their odor or otherwise, will not be accepted.

m. Each lot of pitch compound furnished under this contract will be subject to the tests mentioned, and shall show by these tests the characteristics specified. The tests will be made on samples taken from each lot as it arrives, and the results of the tests on the samples will be considered as characteristic of the lots from which the samples were taken. The rejection of a sample for any reason will cause the rejection of the lot from which the sample was taken. The samples will be taken, by the gallon, from tank cars, by the Engineer or his authorized assistant. If the pitch compound is delivered in barrels, a pint will be taken from each of eight barrels selected at random by the Engineer from each carload of the shipment, and these pints will be thoroughly mixed together to form the sample to be tested.

288.—All pitch compounds, as well as all other materials, used under this contract, shall be subject to the approval of the Engineer, and pitch or other material disapproved by him shall not be used. Any disapproved material shall be removed promptly and replaced by approved material, by and at the expense of the Contractor, on notice to him from the Engineer.

Spreading and Mixing.

289.—When the roadbed, after being prepared and shaped as herein described, does not consist of material in a finely divided state to a depth of at least 6 in., if deemed necessary by the Engineer, it shall be put in such condition by thoroughly plowing, harrowing, and cultivating to a depth of at least 6 in.

290.—On the roadbed, prepared as herein described, the pitch compound shall be spread uniformly by using a sprinkling cart of a pattern approved by the Engineer, or by such other means as may be so approved, and in a quantity not less than 1 gal. nor more than 1½ gal. per sq. yd. of surfacing. Unless otherwise approved in writing by the Engineer, the pitch compound shall be heated to a temperature of not more than 150° cent., and applied at a temperature of not less than 80° cent.

291.—Immediately after an application of pitch compound has been made to a section of road, as above described, the pitched material shall be thoroughly turned over with plows or harrows and mixed so

that none of the unmixed pitch is visible on the surface. A second application of pitch compound will then be made as before. The pitched material will then be mixed thoroughly by the use of plows, harrows, road scrapers, and such other means as may be necessary to insure a complete, uniform, and thorough mixing of the pitch compound and the sandy material of the roadbed. To secure this result, additional pitch compound shall be supplied, where necessary, by the Contractor, in the manner and under the conditions above described. Uniformity of mixing is most important, and must be secured, and pockets of unusually sandy or excessively pitchy material will not be permitted. When finally the roadbed material is mixed satisfactorily, in the opinion of the Engineer, the surface shall be carefully shaped, by raking or otherwise, to the cross-section, grades, etc., shown on the plans, due allowance being made for settlement.

Rolling.

292.—The pitched material shall then be rolled by a "tamping roller" of a pattern approved by the Engineer. This roller shall have toothed concentric rings, or shall be a drum studded with spikes, or shall be of a design which will insure the thorough compaction of the pitched material from the bottom up. The rolling will be continued until the utmost degree of compaction is secured, and until the spikes or teeth of the wheels do not penetrate the surface more than 2 in. During the rolling, and if deemed necessary by the Engineer, the road shall be sprinkled with sufficient water to insure the thorough compaction of the surfacing.

Finishing Coat.

293.—When, in the opinion of the Engineer, the pitched material shall have been sufficiently compacted, as above described, a third coat of pitch compound shall be applied, as hereinbefore described, and in a quantity not less than ½ gal., nor more than 1 gal., per sq. yd. The surface shall then be lightly harrowed or raked, or both, to insure a thorough mixing of the pitch compound with the loose material remaining in the road, and shall be raked and trimmed to the cross-section and grades desired. The road shall then be thoroughly rolled, until firm and hard, with an ordinary steam roller, weighing not less than 5 tons. Any unevenness or depression shall be remedied by the addi-

tion of properly mixed pitch compound and sandy material of a character similar to that in the roadbed, as may be deemed necessary by the Engineer. Should spots in the road show an excess of pitch under this last rolling, sufficient sandy material to absorb the pitch shall be dusted over them carefully and evenly.

Price.

294.—The price agreed to be paid for pitch surfacing shall include all labor and material necessary to do the work herein specified under pitch surfacing, and shall also include the repairs necessary, in the opinion of the Engineer, for a period of two months from the day of the acceptance of the work, which repairs shall be made with the same materials and in the same manner as the construction above specified.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE MAXIMUM WEIGHTS OF SLOW FREIGHT TRAINS.

By C. S. Bissell, M. Am. Soc. C. E.

To be Presented April 21st, 1909.

It is hardly necessary to mention the great and growing importance of the subject of train weights. Study of the problem has given rise to many different forms of investigation which are constantly appearing in the railroad journals and which are indicative of greater refinement in the making up of trains in the future. Most writers have endeavored to treat the subject in its entirety, comprehending the operation of both slow and fast trains, whereas the object of this paper is to present only the case of slow freight trains, and to outline briefly the history of the various steps which led to the conclusions finally reached.

The writer was called upon to formulate some method of estimating operating expenses, over proposed revisions or projected lines not constructed, such as would permit of the intelligent comparison of expenses on two or more projects or routes of haul. The locomotive was of a type weighing, with its tender, 168 tons, of which 173 000 lb. rested on the driving wheels; and the train was to consist of cars weighing 20 tons each when empty, with a capacity of 50 tons. The

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speed was not to exceed 10 miles per hour on ruling grades, and no benefit derived from momentum was to be considered in fixing the train weight. The resistance due to curvature was to be allowed for by compensating the ruling grades at 0.05% per degree of curve.

The first principle of train resistance, namely, that a given train weight confined in a few cars requires less tractive power to move it than the same weight composed of many empty cars, led the writer to a form of equation involving the relation between the dead weight and the lading of the cars, which is evidently the same as if the average car weight or the actual number of cars had been used.

The tractive power of the locomotive was determined as the average amount which would usually be developed within the adhesive power of the driving wheels, considering that atmospheric conditions and steaming qualities remained practically constant at normal values. The tractive power was to be reduced by the proper amount for a constant speed of 10 miles per hour.

With these ideas, which indicated the general form of the equation, the paper* on "Virtual Grades for Freight Trains" by A. C. Dennis, M. Am. Soc. C. E., was consulted, with the result that the train resistance was taken at 4 lb. per ton of train weight for full cars and 9 lb. per ton for empty cars. Also, from this paper, was taken the reduction of 13.7% in the tractive power for the speed of 10 miles per hour.

The hauls in Table 1 are from the records of The Pennsylvania Railroad Company, and represent a fair average of actual train weight for one locomotive of the type described, for a speed of 10 miles per hour.

TABLE 1.

Compensated grade. Percentage.	Gross train weight in tons.
0.300	3 292
0.655	1 898
1.055	1 238
1.260	953
2.130	555

Using the values in Table 1, and increasing the train capacity 10% for overload, as a limiting maximum, the coefficient of tractive

* *Transactions*, Am. Soc. C. E., Vol. L, p. 1.

power was established as 0.232; and the complete expression for tractive power then became 0.232 × 0.863 × the number of pounds on the drivers, or 0.2 × the number of pounds on the drivers, in which the 0.863 is the complement of 13.7 per cent. Introducing now the factor for train resistance, and denoting the ruling grade percentage by g, the general equation is obtained:

$$\left.\begin{matrix}\text{Tons of}\\ \text{train weight}\\ \text{behind tender.}\end{matrix}\right\} = \frac{0.2 \times \text{pounds on drivers.}}{9 - 5\,\dfrac{\text{Weight loaded on train.}}{1.1 \times \text{Capacity of train.}} + 20\,g} - \left\{\begin{matrix}\text{Tons of}\\ \text{locomotive}\\ \text{and tender.}\end{matrix}\right\} \quad ..(1)$$

in which the "Capacity of train" is the sum of the capacities of the cars, and all tons are 2 000 lb. in weight. For the particular case in hand, the weight on drivers is 173 000 lb., and the weight of locomotive and tender is 168 tons; hence, with cars of any class loaded to the maximum, the equation assumes the special form:

$$\text{Net tons behind tender} = \frac{34\ 600}{4 + 20\,g} - 168.$$

Using this equation and the values of g for the hauls given above, from which the coefficient of tractive power was derived, it is found to give the train weights within about the weight of one empty car, except in the case of the fourth haul, which it makes 64 tons too high; but, as the last haul is only 19 tons too high, it is reasonable to assume that the equation represents safe values in the case of this example. Also, it is reasonable to assume that since for empty cars the train resistance is 9 lb. per ton of train weight, the values intermediate between 4 and 9 lb. per ton will be given correctly by the equation for all partial ladings of the train. It will be noted that for a train of empty cars the equation becomes:

$$\text{Net tons behind tender} = \frac{34\ 600}{9 + 20\,g} - 168.$$

Because the purposes for which this equation was derived are usually expressed in terms involving an annual paying tonnage to be moved, the form of the general equation is believed to present special advantages, and from it a table may be formed showing the tons of lading and the number of cars in the train, which is convenient in estimating operating expenses; and the possibility of using it in making up trains is also evident.

The equation has thus far fulfilled the requirements for which it was designed, but suggestions have been made which show that it can be greatly improved in point of accuracy of form and adaptability in making up trains.

One objection to the form is found in the fact that the car resistance per ton varies in a straight line, or, in other words, that it is proportional to a constant increment of lading. Every indication in Nature goes to prove that probably the variation should be in proportion to an increment of lading which is constantly increasing, or to a decrement of similar character; in short, it should represent a curve instead of a straight line when depicted graphically. The grade resistance, 20 g, is evidently mathematically correct, being a simple case of a weight on an inclined plane; but the further suggestion of reducing the value of the tractive power for the grade involved with the dead weight of locomotive, tender, and caboose, has a great advantage in point of accuracy over the method of simply subtracting such dead weight as indicated in the above equation, particularly in cases where the tractive power is determined by a dynamometer car. For general use, it seems advisable to express the relation of the train weight to the number of cars as the average total car weight. It can readily be seen that if the four axles and eight wheels of an ordinary car are identical in size and weight, then the total weight of the train divided by the number of cars is an average car weight, which is a true measure of the resistance per car. If the condition is not realized absolutely, the average car weight will be affected by an inconsiderable amount in the case of an ordinary mixed train.

In view of these considerations, the equation must necessarily be expressed in a form somewhat different from that given above, and, for this purpose, the writer makes use of the following values, which are selected from a number of tests made with a dynamometer car, in which R is the resistance of a train, in pounds per car, and W is the average weight per car, in tons of 2 000 lb., for a speed of 10 miles per hour:

TABLE 2.

Point.	R (pounds).	W (tons).
A	160	20
x	195.7	40.2
y	199.0	42.8
B	216.0	72

Referring to Fig. 1, upon which the points, A and B, are located, it is clear that the resistance per car will decrease from B to A, and will become zero when the number of cars becomes zero. Hence the line of variation must pass from B through A and through the origin

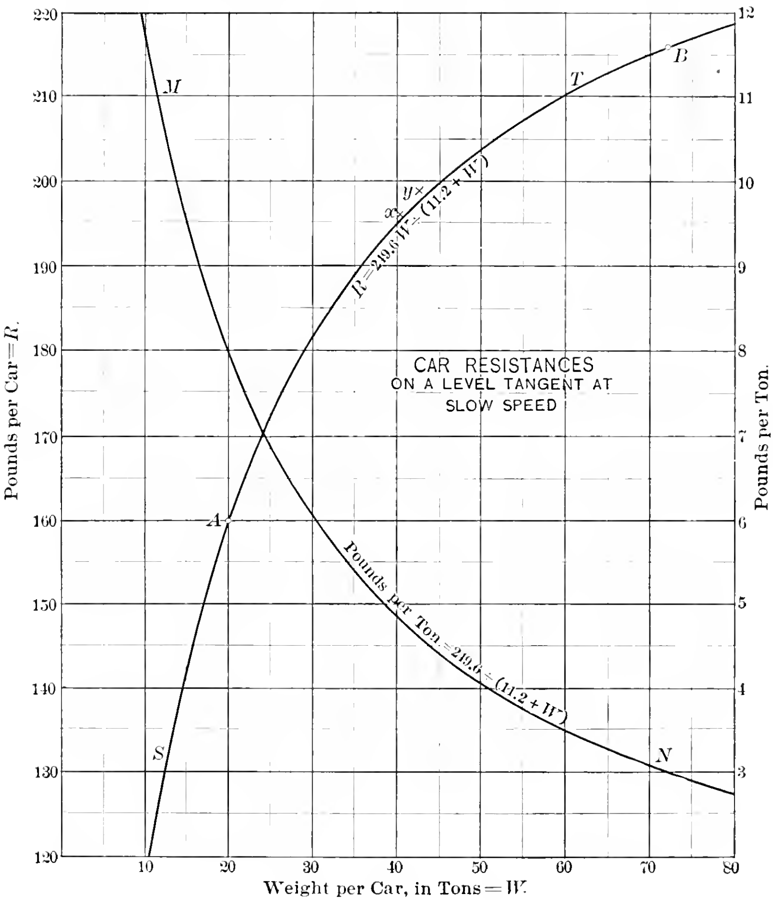

Fig. 1.

of co-ordinates, zero. It must, therefore, be a curve; and such a curve is represented by the equation:

$$R = \frac{a\,W}{b + W};$$

its particular form for this case being,

$$R = \frac{249.6\,W}{11.2 + W}.$$

which is the line, $S\ T$, in Fig. 1. Plotting the points, x and y, it is seen that they agree closely with the values given by the equation, thus confirming the assumption that the resistance per car varies as a curved line of this character.

Dividing the values of R by the corresponding values of W, and plotting the results to the scale on the right of Fig. 1, develops the curve, $M\ N$, of pounds per ton of train weight.

It is now possible to formulate the equation:

$$\text{Tons of train weight behind tender} = \frac{P - 20\,g\,m}{\dfrac{a}{b + W} + 20\,g} \quad \ldots\ldots\ldots (2)$$

in which P = the pounds of tractive power at the drawbar; m = the weight, in tons, of the locomotive and tender; a and b are constants; W is the average weight per car, in tons; and g is the grade, in percentage. The tractive power, P, is here reduced by $20\ g\ m$ (where the weight of a caboose may be included in m if desired), instead of subtracting the dead weight, m, after the division has been made. Substituting the known terms taken from Tables 1 and 2 and those derived for Fig. 1, an average value of P is found to be 32 100 lb., and therefore the equation for train weight for this particular case becomes:

$$\text{Tons of train weight behind tender} = \frac{32\,100 - 3\,360\,g}{\dfrac{249.6}{11.2 + W} + 20\,g}.$$

This equation does not agree with the values in Table 1 as closely as do the results from Equation 1, but it is more rational in form, and is adapted to use in connection with records taken with a dynamometer car, from which the tractive power, P, and the constants, a and b, can be readily determined by experimental trials for any particular forms of locomotives or cars.

For the purpose of making up trains, an equation similar to the above can be expanded into a table. Thus, for any particular division of the railroad, the ruling grade, g, is known; assuming it to be 0.5%, for example, the equation becomes:

$$\text{Tons of train weight} = \frac{30\,420}{\dfrac{249.6}{11.2 + W} + 10}.$$

From this may be prepared for all car weights a form similar to Table 3, which shows only three values of average car weight. The

reduction for temperature, of course, must be determined by past experience. A similar table, prepared for each class of locomotive on the division, can be used for making up trains by a yard-master of very mediocre intelligence. The process of taking the continued sum of the car weights until the total indicated in the table is approximately reached, and then dividing the result by the number of cars to obtain the average car weight, is a simple one. The difference of a few cars more or less is then made up, and the total train weight of the completed train is finally checked by a repetition of the process.

TABLE 3.

Average car weight, in tons.	M. & N. Division—Locomotive Class H.				
	Train Weights for Various Temperatures.				
	Summer.	45° — 25°	25° — 5°	Below 5°	Emergency.
40	2 046	1 840	1 636	1 535	1 023
41	2 058	1 852	1 646	1 544	1 029
42	2 071	1 864	1 657	1 553	1 036

This example of the manner in which trains may be made up is given to illustrate the practicability of dispensing with a "car factor," consisting of a predetermined number of tons to be added to each car in the train. Such a car factor is in reality a fictitious weight, and its use introduces the element of a "straight-line variation" in reaching the final result, which is questionable in point of accuracy. The following quotation is taken from "Tonnage Rating," an article by Mr. F. W. Thomas*:

Mr. Thomas says:

"The most difficult problem after the rating has been ascertained and proven, is to express the rating intelligently and in such a form that the dispatchers, yard-masters, foremen or switch crews and conductors can understand and easily interpret the rating sheets.

"The most difficult thing to impress upon those interested is the fact that the rating is often governed by the number of cars in the train; the greater the number of cars, the greater the rolling resistance. In the eyes of the average train master and dispatcher a thousand tons, whether confined in twenty cars or in fifty, is a thousand tons."

* * * * * * *

* *American Engineer and Railroad Journal*, April, 1907.

"You will note * * * that the maximum rating is shown in cars weighing fifty tons, and for every car added to the train above this given number of cars a reduction from the maximum rating must be made of from four to five tons. * * * I cannot say that this reduction is based on any fixed rule, beyond, as mentioned above, that it is the fruit of long investigation and a series of tests * * * ."

The writer thinks that Mr. Thomas intended to say, in the instance above, "the greater the number of cars the greater the rolling resistance per ton of train weight," which is usually difficult to make clear.

As a matter of fact, the determination of the car factor is sometimes based on a "fixed rule," or rather a carefully wrought out series of experimental tests put into a rational mathematical form. This form usually is developed by equations, some of which represent straight lines, but whether or not this be true, the unfortunate fact remains that the use of such a car factor, when derived, constitutes a constant increment, thus introducing a form of straight-line variation into the calculation.

It may be asked, what objection can be made to such a variation, and in answer the writer appeals to the reason of those who are students of the problem here presented. Natural forces represented graphically, almost without exception, are forms of curves. The construction of a train is artificial, but the forces developed by the movement of the train are natural in their characteristics; and continued study brings with it the growing conviction that the graphics in this problem should be curves rather than straight lines. Granting this, the use of the car factor must be dispensed with, and a rigidly curvilinear method or equation must be formulated. If we conceive of a level railroad tangent as an arc of 4 000 miles radius, the center of which is the earth's center, then Equation 2, as given above, is strictly curvilinear in its components and in its entirety; but if this conception appears to violate the sense in which the quantity "20 g" is used, there still remains the argument that grade is an artificial and not a natural element of force in this problem.

The economic issues dependent upon a true determination of maximum train weights, especially for slow freight trains, are of such grave importance that the writer feels justified in stating that any criticisms or suggestions which will throw more light on the subject will be welcomed by the majority of railroad men.

The data upon which the foregoing developments are based are believed to represent normal conditions. They are the results of many tests, and of careful selection and reduction. In every case, however, they apply only to the slow speed of from 10 to 7 miles per hour. The equations are given to show only the form of variation, since it is apparent that the constants in them must be derived from experimental tests.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE PURIFICATION OF GROUND-WATERS CONTAINING IRON AND MANGANESE.

Discussion.*

By Messrs. W. C. Lounsbury, Arthur L. Terry, Jr., Philip Burgess, A. Elliott Kimberly, and Alexander Potter.

Mr. Lounsbury.

William C. Lounsbury, Esq.† (by letter).—Mr. Weston mentions the difficulties which may be encountered in removing iron from underground supplies. The plant at Superior, Wis., has certainly had its share of troubles arising from this difficulty, and a brief account of the somewhat peculiar conditions and experiences may prove of interest, in the light of Mr. Weston's suggestions.

Superior is the second city in Wisconsin, and is located at the head of the Great Lakes. A narrow strip of sand, some 7 miles long and a few hundred feet wide, runs parallel to the Wisconsin shore, about a mile from the city and mainland. Lake Superior washes the outer shore of this sandy stretch, and Superior Bay, the inner shore. This land, called Minnesota Point, is principally made up of a fairly coarse sand, mixed with considerable quantities of fine particles of metallic iron. In the formation of the point, logs, peaty matter, clay, and débris have undoubtedly been incorporated with the sand, so that the clean sand of the surface shows no indication of what may be underneath.

About ten years ago, the Water Company decided to draw its supply from wells to be located on Minnesota Point, along the shore of the lake. By this means, it was thought, an excellent supply of

* This discussion (of the paper by Robert Spurr Weston, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for December, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chemist and Superintendent of the Water-Works, Superior, Wis.

Mr. Lounsbury.

filtered water could be obtained. Accordingly, eighty odd wells were jetted to a depth of about 14 ft. These wells were 6-in. wrought-iron pipes, carrying Cook brass strainers, and were spaced about 17 ft. apart, each being separated from the main suction by a valve. The tops of the wells were below high-water level, and the ingress of the water was some 25 ft. below this. The shore line varies, but the wells are approximately 20 ft. from the lake and 400 ft. from the bay across the point. During high water, the water washes over the wells, but, on account of the ice in winter, and in times of low water, the free water of the lake may be several hundred feet away. The water derived from these wells was extremely disappointing. It was harder than the lake water, although not as hard as the bay water; moreover, it carried considerable iron.

In 1901, G. C. Whipple, M. Am. Soc. C. E., made an analysis of the water, as follows:

Total solids	78
Nitrogen as:	Total.
Albuminoid ammonia	0.066
Free ammonia	0.016
Nitrites	0.001
Nitrates	0.04
* * * * * *	
Iron	1.50
Hardness	48.5
Alkalinity	46

The character of the water has remained practically unchanged since. The water as described was used under direct pressure to the city for several years, but caused much trouble. The iron settled out in the pipes; *Crenothrix* appeared in great quantities; and all the characteristic objections to iron manifested themselves in a marked degree. The water was particularly disagreeable because it became highly colored, after being drawn from the wells, and this highly-colored water was confounded, by the public, with the objectionable, highly-colored water of the bay, under which the intake passes. This latter water, it may be stated, carries considerable organic matter, leached from tamarack swamps, and also iron in solution. Its color varies above 100.

In 1899, Allen Hazen, M. Am. Soc. C. E., designed and constructed an iron removal plant, for the local company. This plant* consists, essentially, of an aerating device, by which the water showers through holes in iron trays, and falls as spray into a well beneath. The trays are enclosed in a skeleton house, the sides of which are

* Described at length in *Engineering News*, February 21st, 1901, p. 141.

Mr. Lounsbury. shutters, arranged so as to give free access of the air to the water. From here, the water flows to covered sand filters, and is filtered at a rate of about 5 000 000 gal. per acre, per day. This method of treatment at once gave improved conditions, and the effluent has never since supported any growth in the mains, or deposited iron in them to any extent. Strangely enough, however, the water showed, by analysis, an iron content of 0.7, or 0.8, or even 1.0 part of iron in the filtered water, and an average color of 30, varying from a minimum of 25, to a maximum of 40. To reduce the iron still further, the aerator was heightened, so that more air came in contact with the water. This gave no beneficial results.

The carbon dioxide content is fairly constant, at 1.2 parts. Aeration as used reduces this to an amount of from 0.4 to 0.6 parts.

The writer's experience with the plant commenced in May, 1906. It was very important, at that time, to remove the color of the effluent, on account of local political conditions. In studying the matter, and in the light of a few bottle experiments, the plant was run for several days without the use of the aerator. The result was an immediate reduction of color and the iron content in the effluent. The iron was almost completely precipitated, and the color was reduced so much that the water was perfectly satisfactory. The results are indicated on the diagram, Fig. 9, which shows the decided change that takes place when the aerator is put into use, or is by-passed.

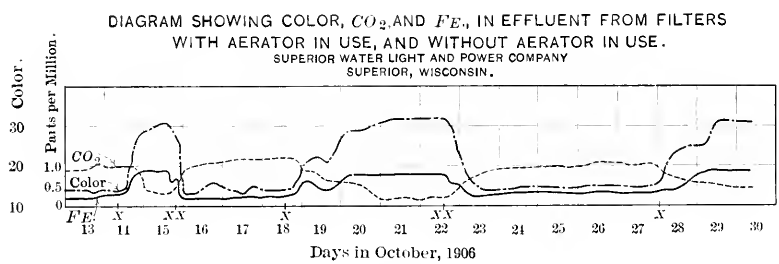

FIG. 9.

In Fig. 9, the time of putting the aerator into operation is marked with a cross, *X*, and the time of by-passing is marked with a double cross, *XX*.

The samples of water were collected at the pumps, so that the effect is seen several hours after the time marked. Samples were analyzed at least once in 4 hours, during the entire period shown. The slight increase of color on October 16th is due to placing in operation a filter previously filled with aerated water.

The analyses were all made by the writer, and are expressed according to the American Public Health Association standards.

The most obvious thing done by this treatment, or lack of treatment, was to allow the filtration to take place in the presence of a greater amount of carbon dioxide, and to pass the water to the filters without violent shaking. There is always an excess of air in the water, through leakage of the system and at the pumps, therefore the writer is inclined to think that the amount of oxygen present does not play an important part. This condition was exactly what had been found in the laboratory. Bottles of the same raw water had been taken. Into one was run carbon dioxide, charging the water under the atmospheric conditions prevailing. The other bottle was simply stoppered. Both were well shaken, and allowed to stand. It was found that the one containing the excess of carbon dioxide always precipitated its iron readily, without attaining the yellow color the control bottle did. Other bottles, after having stood a year or more, have invariably deposited most of the iron, although not always losing their color. The quantities of precipitant deposited on the filters when operating without aeration, was so great that the filters were totally inadequate to handle it. Ordinarily, with aeration, a filter runs from 7 to 9 days; without aeration, one dies in 3 or 4 days. Mr. Lounsbury.

Non-aeration has been tried, at different times, for more than two years, and has invariably produced a satisfactory effluent, having a color of from 15 to 18 and an iron content of never more than 0.5 part, and frequently as low as 0.3. These experiments have covered ranges of water temperature from 34 to 60°, and air temperature of from —30° to +90°, the rates of filtration varying with the varying consumption of the city, and the loss of head due to the age of the filters. A proportion of lake water has also been admitted, the alkalinity of which was 40, and the hardness 37, and containing no iron; but always with the same result.

Mr. Weston reports a manganese content of 0.12 part in this water, and possibly this plays an important part in the trouble. Whatever the explanation may be, it is certain that the Superior water has been so constant in composition for the past few years that the iron can be readily and easily precipitated, and the color can be kept from forming, by filtering without aeration. Possibly Mr. Weston can explain just why this is so.

Arthur L. Terry, Jr., Jun. Am. Soc. C. E. (by letter).—It may be of interest to those undertaking the purification of ground-water to know that a deferrization plant of 700 000 gal. daily capacity was installed for the Government by Hungerford & Terry at Fort Hancock, Sandy Hook, N. J., more than a year ago. Mr. Terry.

The original water supply of the fort consisted of a series of well points driven into the sand about 6 ft. below the surface. These gave a very good water until the consumption reached about 75 000 gal. a day, when evidences of sea-water began to be found, and increased to

Mr. Terry. such an extent that four years ago two deep wells were installed, one 450 ft. and the other 750 ft. deep. The water from both these wells was very soft and good in every way, except for the iron content, which reached about six parts per million in both wells. The deeper well flowed at an elevation of 15 ft. above tidal water, discharging about 20 gal. per min. The water contained considerable quantities of hydrogen sulphide. Like most deep-well waters, it was clear when first discharged, but in a short time developed considerable turbidity. An attempt to use this water without treatment resulted in so much dissatisfaction that the authorities reverted to the old supply. After a year, two settling reservoirs were built and air-lifts were installed, the idea being that aerating the water would oxidize the iron; and that the settling reservoirs, which gave about 5 days' sedimentation, would permit all the iron to be removed. This plant was completed about 2½ years ago, and put into operation, but it was found that while the oxidation of the iron appeared to be very rapid, and that this precipitated iron settled with considerable rapidity, yet about 30% of the iron remained in solution. At about that time a proposition was made by Hungerford & Terry to install a plant using lime as a coagulant, utilizing the settling reservoirs and supplementing the plant with mechanical filters. This proposition was accepted, and the plant was installed substantially as follows:

The two wells, each of which would discharge 250 gal. per min. when using the air-lift, were conducted into an orifice box containing two adjustable orifices. The large orifice is set so that the head of water varies from 1 to 2 ft. over the orifices, depending on the discharge of the wells. The area of the large orifice is about 17 sq. in., that of the small orifice is about ¼ sq. in., and both are placed at the same elevation, so that the discharge of one is approximately proportional to that of the other, regardless of the fluctuations in head. The small orifice discharges into a lime saturator. This saturator consists of a conical vessel, at the bottom of which is placed about a bushel of freshly slaked lime. The incoming water from the small orifice is discharged at the bottom of the saturator, and, to aid in a thorough mixing of the applied water with the lime, a small air line is run from the air compressor, which produces a constant ebullition part way up the saturator. A conical hood, terminating in a pipe reaching above the water level, carries the air to the surface after it has performed its duty of mixing the lime and water. All below this hood is a violently agitated milk of lime, while above and outside the hood there is a solution of lime gradually clearing as the surface is reached. The outlet of the saturator is on the side at the top, and when raw water enters the saturator it displaces an equivalent amount of a clear saturated solution of lime, and in its turn is converted to milk of lime. The quantity of water applied to the saturator is ordinarily

about 7 gal. per min., and this, of course, displaces a corresponding quantity of lime water. This lime water is applied to the raw water from the large orifice as it enters No. 1 reservoir. Coagulation is almost instantaneous, and is complete. Mr. Terry.

The water takes about two days to pass through the two settling basins, and, although apparently clear, a very considerable amount of iron is still left. This fact has been determined, not by analysis, but by observing the wash-water from the filters, the latter requiring washing twice a week.

The filters are mechanical filters of the pressure type, each having a capacity of 360 000 gal. per day of 24 hours, or 720 000 gal. per day for the two units.

The lime required for this plant costs only 80 cents per million gallons, and does not seem to vary with the seasons, for the lime feed has not been changed since the plant was first installed. That the correct quantity of lime was being used was determined by testing the alkalinity of the water with phenolphthalein. When the filtered water shows a faint pink color with phenolphthalein, the removal of the iron is almost complete, although a pink reaction in the water before filtration also indicates very satisfactory results. It is a rather remarkable fact that the water reacts strongly to phenolphthalein before filtration, and yet shows no reaction afterward.

The following report of the water comes from the office of the Surgeon-General of the United States Army:

"The first [sample] is the water as it comes from the wells. It is a turbid water of a very faintly alkaline reaction, odorless, of a yellowish color, due to the marked amount of suspended ferruginous matter. This settles on standing to an appreciable deposit, leaving it a clear, colorless supernatant fluid.

"The second is the water after treatment through the filtration apparatus. It is absolutely clear and colorless, is faintly alkaline in reaction, contains an almost inappreciable sediment on standing for many hours, and is without odor. There is no visible charring or carbonization in the residue on its incineration. There thus appears a great improvement in the water by the removal of what small amount of organic matter it contained, together with practically all of the iron, and at the same time the amount of the mineral matter was reduced in its total by one part per million. The actual amount of iron present in this sample is 0.0140 part per hundred thousand, while its hardness (Clark's) is 4.54 per hundred thousand.

"Edwin R. Hodge,

"Chemist in charge."

Unfortunately, the Government did not report the analysis of the raw water as it came from the wells, but another analysis, made at the same time, showed the total iron to be six parts per million.

Before the receipt of the analysis of this sample, the contractors

Mr. Terry. requested that another sample be taken by the Government for analysis, they believing that a still higher efficiency could be shown by increasing the lime to such a point that the filtered water would react slightly with phenolphthalein. This was forwarded to the War Department, and the following analysis was returned:

"Contained iron 0.0088 part per hundred thousand.

"Total solids: Treated water, 49.50. Raw water, 82.50 parts per million.

"EDWIN R. HODGE,
"Chemist in charge."

Experience with this plant indicates that the lime treatment, when installed as described, is cheaper than any other process, not excepting aeration. To be sure, an air-lift is used in conjunction with this plant, but that is only for 6 hours per day, and, during the remaining time, the well flows and there is no aeration of the incoming water, the result being obtained by the lime alone.

When it is considered that this plant has no skilled supervision, but is run by employees whose sole duty (with regard to the plant) is to wash the filters twice a week, and slake and discharge into the saturator a few bushels of lime; and, also, that the plant always maintains an almost total reduction of the iron content, this method of deferrization is worthy of notice. Those who have tried to introduce clay or milk of lime into water with any degree of accuracy know what a difficult and unsatisfactory proceeding that is.

Taking the experience at Reading, reports state that the water was hardened very considerably. On the other hand, the Fort Hancock plant showed virtually no change in the hardness, except on the occasion of the second analysis, when the lime was increased to a point at which softening began; but then, instead of increasing the total content of the water, there was a reduction of nearly 50% in the total solids.

Referring to aerating towers of coke or other substances, it seems, from the knowledge at hand, that elevating the water and cleaning such towers, to say nothing of the interest on the investment, would make the cost greater than treatment by lime, and there can be no doubt that a lime-treated water will contain far less iron than one which relies merely on aeration.

To carry the matter further, it even seems that some of the abnormal conditions which have arisen at Posen and other places might be handled successfully by the lime process, followed by subsequent filtration. A slight excess of lime applied to a water containing a large amount of humus is a most energetic coagulant, and it certainly should be far more effective than the present methods.

The only objection that the writer can see to the use of lime is in the uncertain methods by which it has been applied in the past. An

excess of lime, under ordinary conditions, is an unpleasant feature, but when applied in the manner described, an excess is an impossibility if the plant is designed properly. The worst that could occur would be a deficiency due to inattention of the operative in not placing slaked lime in the saturator. Mr. Terry.

Mr. Burgess. PHILIP BURGESS, ASSOC. M. AM. SOC. C. E. (by letter).—This excellent paper will doubtless prove of great interest to all engineers engaged in the installation and operation of public water supplies. The information contained tends to make clear many of the problems encountered in the search for an economical water supply of satisfactory quality. The question of approval of ground-waters has been frequently referred to the Ohio State Board of Health, and the writer has made many examinations of water from ground sources, waters which have contained frequently such large quantities of iron as to be unsatisfactory for domestic use without purification.

In the Middle West it is frequently necessary for the engineer to judge between the advantages of a supply from ground sources and one of purified surface-water. While the former is frequently clear and potable, it is sometimes so objectionably hard that a surface-water would be more desirable. The double cost of pumping, generally necessary with ground-water supplies, sometimes makes the cost of operation less than that of a filtered surface-water only to the extent of the cost for a coagulant necessary when the water is purified with mechanical filters. The costs of construction shown by Mr. Weston in Fig. 1 are evidently based on slow sand filters. The average cost of thirteen mechanical filter plants of varying capacities in Ohio, including low-service pumps, coagulation and sedimentation basins, filters, and clear wells, is about $14 000 per million gallons, while the cost of operating the purification devices averages about $6 per million gallons. These figures show that the cost of securing a filtered water supply of surface origin is frequently not much greater than that necessary to procure a satisfactory ground-water. Moreover, the latter is frequently so hard that the total expense to the consumer is considerably greater than is the case with the softer surface-water. Cases are not infrequent where a ground-water supply has become dangerously polluted without the knowledge of the operator until a considerable quantity of badly polluted water has been pumped to the consumers. In the writer's opinion, such conditions are very rare with a well-constructed and operated filter plant.

One feature relative to the distribution of iron-bearing waters is little touched upon by Mr. Weston, and that is the great variation in the quantity of iron in water from different strata in the same locality. The writer has in mind, especially, two wells at Newton Falls, Ohio, one 58 ft. deep containing 3.0 parts of iron per million, and the other, not 50 ft. distant, 35 ft. deep, and containing 20.0 parts of iron per

Mr. Burgess. million. Similar variations have been found at different depths in other parts of Ohio.

Mr. Weston's discussion on the design of iron-removal plants is so general that it will hardly serve as a reference for engineers who have to study the problem of the purification of a specific ground-water. The scarcity of the data on such plants in America and the great variations in the character of the water to be treated, doubtless account for the absence of more specific statements. The following data are presented in order to show what has actually been accomplished by iron-removal plants at Garrettsville and at Shelby, Ohio.

The water supply for Garrettsville is derived from ten driven wells varying in depth from 41 to 70 ft.; also from a shallow dug well or flowing spring. The wells and spring are in a ravine just above the confluence of two small streams. The wells pierce, first, a layer of yellow clay of varying depth, and second, a layer of white sandstone rock, from 40 to 60 ft. in depth and of nearly pure silicious material. The lower 8 ft. of the sandstone strata are of a conglomerate nature and comprise the water-bearing material from which all the wells derive their supply. The water is of generally satisfactory quality for domestic use, except that its iron content is about 2.0 parts per million. It was for the removal of iron that a purification plant was constructed in 1906. The devices provided for the removal of iron comprise an aerator, a sedimentation basin, and a mechanical filter.

The aerator consists of an umbrella-shaped device on the discharge of a low-service force main. It is 4 ft. in diameter and approximately 5 ft. above the level of the water in the basin. When the plant is in operation, water from the wells is lifted by the low-service pump, and, at the point of discharge into the basin, falls back on this umbrella-shaped device which breaks it up into a fine spray. The sedimentation basin is divided into four compartments, about 12 ft. square and 9 ft. deep. Its total capacity is 39 000 gal. The filter is of the gravity mechanical type, and is a rectangular concrete tank with a total area of 96 sq. ft. The filtering material comprises 12 in. of graded gravel and 36 in. of sand of about 0.3 mm. effective size.

During a recent inspection by the writer, the consumption at Garrettsville was about 25 000 gal. daily and the rate of pumping on low service about 2 000 gal. per hour. This gives a period of sedimentation of practically 20 hours and a rate of filtration of approximately 21 000 000 gal. per acre per day. The settling basin is cleaned about once every 3 months, and it is found that practically all the sludge accumulates in the first compartment, where there is generally a deposit about 1.5 in. in depth. Every 3 days it is necessary to back-flush the filter, which is accomplished by applying filtered water from the reservoir. Washing is accomplished in about 10 min., and requires about 15 000 gal. of water, equivalent to about 20% of the total quantity filtered.

The consumers appear to be very well satisfied with the quality of the water and the plant is operated without difficulty, but at considerable excess cost due to the unusually large quantity of wash-water necessary. This could be reduced by improving the washing system of the filter. Mr. Burgess.

The water supply for Shelby, Ohio, is obtained from ten 6-in. driven wells in the alluvial plain bordering Black Fork Creek. The wells vary in depth from 40 to 60 ft., and are from 200 to 300 ft. from the creek. They pierce the following material, in the order stated: Loam, clay, hardpan, gravel, slate, sandstone rock, and water-bearing gravel. The thickness of the strata is not known, but it is said that the slate and sandstone are both thin. The fact that leaves and twigs were found in some of the wells at considerable depths indicates that, in prehistoric times, the ground from which the water is obtained was of a swampy character.

During pumping, water generally stands in the wells about 19 ft. below the surface of the ground, and it was not lowered during the recent exceptionally dry spell. This would indicate that the water is derived from ground sources, and not from the creek. Although objectionably hard, the water is of generally satisfactory quality for domestic use, except that its iron content is 6.6 parts per million.

The water was used without purification for eight years, but considerable trouble was caused by the precipitation of the iron in the mains, and a purification plant was built in 1905. This plant comprises an aerating tank and two horizontal pressure filters. The aerating tank is of steel, 120 ft. high, and 8 ft. in diameter, and is placed concentrically within the old stand-pipe, which is 130 ft. high and 16 ft. in diameter. In the top of the tank are provided steel plates perforated with 1-in. holes 2 in. apart. The plates are 2 ft. apart, staggered, and each covers one-half the cross-section of the tank, so that the actual distance between the plates is 4 ft.

The filters are horizontal steel tanks, 24 ft. long and 8 ft. in diameter. In the bottom is a collecting system, above which is placed 18 in. of screened gravel and 18 in. of crushed quartz sand. The latter will all pass through a screen of 16 meshes per inch and be retained on a screen of 30 meshes per inch. Its effective size is approximately 0.75 mm.

The present daily consumption at Shelby varies from 250 000 to 300 000 gal. Water from the filters is pumped directly into the mains, and the plant is operated 24 hours. The force main from the low-service pump is carried up inside the outer compartment of the stand-pipe and discharges into the aerating tank, in which the water is generally maintained at least 10 ft. below the top. The aerated water flows by gravity through the filters, and is pumped into the mains. The average period of sedimentation in the tanks is about 4 hours, and the filters are operated at about 30 000 000 gal. per acre per day.

Mr. Burgess. They are washed once a week, using from 20 000 to 40 000 gal. of water. The initial and maximum losses of head are 2 and 6 lb., respectively. The stand-pipe is flushed out about once a month, but little sediment is removed.

On January 14th, 1909, the writer, in company with Mr. C. F. Long, of the Laboratory Department, made an examination of the Shelby plant. During this examination it was the purpose, as far as possible, to make the necessary analyses in the field. The temperature, turbidity, color, dissolved oxygen, alkalinity, iron, oxygen consumed, and total bacterial content were determined at the plant, while the other analyses shown in Table 19 were determined on samples shipped to the laboratory of the State Board of Health. The examination covered 18 hours' operation of the plant, but indicated little variation in the character of the water at the several sampling points.

TABLE 19.—RESULTS OF ANALYSES OF SAMPLES COLLECTED AT SHELBY, OHIO, ON JANUARY 14TH, 1909.
(Parts per Million.)

Determinations.	Raw water.	Aerated water.	Filtered Water.	
			At filter.	In mains.
Temperature, in degrees, centigrade	10.9	10.1	9.9	11.5
Turbidity	0+	60±	10±	5+
Color*	3	......	45	25
Free CO_2	48	28	24.5	22
Dissolved oxygen				
(Parts per million)	0	7.8	7.7	7.5
Percentage saturated	0	69	68	69
Alkalinity	263	248	241	240
Incrustants	214	220	215	220
Total hardness	477	468	455	460
Total solids	713	......	713	......
Loss on ignition	73	......	82	......
Total iron (Fe)	6.5	6.7	0.85	0.60
Magnesium	31	......	31	......
Chlorine	11.8	12.5	11.5	13.5
Oxygen consumed	3.65	2.58	2.13	1.90
Nitrogen as				
Free NH_3	5.80	......	0.180	......
Alb. NH_3	0.112	......	0.104	......
Nitrites	0.002	......	0.002	......
Nitrates	0	......	0	......
Bacteria per cubic centimeter	1	......	1	......

* On unfiltered samples.

The analyses of the raw water indicate that it was clear, hard, of high iron content, and contained no dissolved oxygen; its free CO_2, free ammonia, and oxygen consumed were all high, while nitrates were absent. The water was practically sterile.

The analyses of the aerated water indicate a considerable change by aeration, notably the increase in turbidity due to the oxidized iron; the elimination of considerable free CO_2 displaced by dissolved oxygen; and a reduction in alkalinity. The total iron content was not reduced,

but the latter was in a changed form, and was chiefly in suspension as an oxide. Mr. Burgess.

The analyses of the filtered water indicate that the filters removed most of the suspended iron, so that the final product contained but 0.85 part per million. The color determinations indicate that the residual iron was in suspension. The samples from the mains indicate a further reduction, or iron sufficient to cause an accumulation of deposits in the mains and a slight staining of plumbing fixtures.

In the writer's opinion, the efficiency of the filters could be considerably improved were the filtering material replaced by a greater depth of finer-grained sand having an effective size of about 0.3 mm. In general it may be said, however, that the purification devices have served to eliminate the objectionable features so that consumers are now generally satisfied with the quality of the water.

An interesting point brought out by the analyses is the considerable reduction of alkalinity by the aeration of the water and the oxidation of the iron, a fact which tends to show that the latter is present in the form of a carbonate. There is also a striking reduction of free ammonia, which apparently goes off as a gas, as the filtered water contains no nitrates. Further information on these points is very desirable.

A. Elliott Kimberly, Esq.* (by letter).—While there are many factors which favor the use of water supplies from ground-water sources, it is to be remembered that ground-waters in the Middle West are usually hard and, from this standpoint, objectionable. Methods of water filtration for the removal of suspended matters and bacteria have reached such a state of efficiency that, ordinarily, it is feasible and economical to consider thoroughly the use of a surface-supply, rather than derive the supply from ground-water sources. This is particularly true where the surface-water is soft and the ground-water hard. The small increase in operating charges incidental to the operation of a filter plant is but slightly more than the actual money loss incurred by the consumer of the hard ground-water. The argument is still more forcible where deferrization is necessary, and, before deciding on the use of a hard, ferruginous ground-water, all reasonable means should be investigated for obtaining a softer supply from surface sources. Mr. Kimberly.

The presence of iron in ground-waters often prevents their use as sources of water supply, unless means be provided for removing the iron, and to this important subject Mr. Weston has certainly contributed a mass of valuable data.

The writer was at one time engaged in studying the water at Reading, Mass., under the direction of Mr. H. W. Clark, where the difficulty encountered in removing the iron was attributed, not so

* Asst. Engr., Ohio State Board of Health.

Mr. Kimberly. much to the presence of organic matter, as to the fact that the iron was present in the form of a sulphate, or, more correctly, was accompanied by the sulphate radical, SO_4, rather than by the carbonate radical, CO_3. To this fact was ascribed the greater ease of iron removal by aeration possible in the cases of the Provincetown and the Watertown waters.

It is interesting to note that it is not the nature of the acid radical, but the presence of organic matter, as Mr. Weston's researches clearly show, which prevents the precipitation of the iron after it has been oxidized.

As is well known, the removal of clayey matters in suspension is readily accomplished in water-filtration practice by sulphate of alumina. The use of a clay-bearing, ferruginous water as a precipitant for domestic sewage, as at Hanley, England, suggests several interesting Ohio experiences.

The sewage of Sebring, Ohio, in volume about 46 000 gal. in 24 hours, flows in an open ditch into Sulphur Creek, which has its rise in an abandoned coal mine. In dry weather, the creek flow is at the rate of about 28 000 gal. in 24 hours. The crude sewage is essentially domestic, is ordinarily weak, and contains a predominance of sink wastes and soapy matters. The Sulphur Creek water is an acid mine waste which contains free mineral acid, very large quantities of iron, high sulphates, and other constitutents peculiar to mine drainage. Analyses of the Sebring sewage and of the Sulphur Creek water are given in Table 20.

About 150 ft. below the confluence of the two streams, marked coagulation is visible. The velocity of the current is very slight, and, for a mile or more from the point of confluence, the stream is practically a sedimentation basin under continuous operation. At its outlet, one mile below the confluence, the supernatant water shows practically no turbidity and, besides being substantially free from iron and free acid, contains but little evidence of the marked organic pollution it has received.

The bottom of the stream, however, is a mass of putrefying sludge with a slight surface coating of ferric hydrate, surmounting a mass of black, nauseating, and foul deposit. These conditions are illustrated in Table 20. They indicate the effect of semi-colloidal sewage upon the iron present in mine wastes, and are another example of the deferrization of ground-waters by means of colloids.

Several further illustrations of the use of negatively-charged clay particles in hastening coagulation occur to the writer. At Warren, Ohio,* the Mahoning River water is purified by mechanical filters of the wooden tub type, and, at times, the turbidity of the raw water is so slight that clay is used to start the coagulation. At Newark, Ohio,*

* See forthcoming report of Ohio State Board of Health on Water and Sewage Purification Plants.

Mr. Kimberly.

during certain seasons, the raw water is derived chiefly from the ground, and is so free from turbidity that it cannot be coagulated with sulphate of alumina unless clay be used. Clay has also been used at Harrisburg, Pa.,* and Ithaca, N. Y.,† to assist in obtaining a satisfactory floc under certain conditions of the raw water.

TABLE 20.—ANALYSES OF SEBRING SEWAGE AND SULPHUR CREEK ABOVE AND AT SEVERAL POINTS BELOW THEIR CONFLUENCE, ILLUSTRATING THE PRECIPITATION OF IRON-BEARING WATERS BY COLLOIDAL MATTER.

Sampling points.	Parts Per Million.							Bacteria per Cubic Centimeter.
	Turbidity.	Alkalinity.	Nitrogen as Free Ammonia.	Chlorine.	Dissolved Oxygen.	Iron (Fe).	SO_4	
Sulphur Creek..........	0+	—62*	0.8	23	0	50	1 044†	135
Sebring sewage.........	500	350	16.0	94	0	....	....	600 000
150 ft. below confluence.	250	154	13.0	59	0	40	808	8 000
1 mile below confluence.	5±	24	3.0	38	0	2	495	5 300

* Free sulphuric acid present. † Incrustants.

The question of iron removal is of special significance and importance in connection with problems of water supply for small villages. Recognizing the allowable limit of 0.5 part per million of iron in a potable water, what means are available and should be recommended for removing iron from ground-waters when present in objectionable quantities?

In reviewing the foreign and domestic deferrization plants, as described by Mr. Weston, it is apparent that there are several distinct types:

- Aerators, sedimentation tanks and open filters (mechanical or slow sand);
- Aerators and pressure filters;
- Air-lift pumps, head tank, pressure filters, clear well or direct connections to mains.

Available evidence as to the best practice in the design of deferrization plants appears to be somewhat fragmentary, and it is apparent from Mr. Weston's paper that the design depends primarily on the character of the raw water, which should be carefully investigated. It is obvious that aerating devices and pressure filters which, in the absence of organic matter, are successful in removing the iron with

* Harrisburg, Report of Board of Commissioners of the Water and Lighting Department, 1907, p. 24.

† "Operating Results of the Water Purification Plant at Ithaca, New York." *The Engineering Record*, Vol. 57, 1908, p. 673.

Mr. Kimberly. sufficient thoroughness, show somewhat high operating costs, owing to the heavy loss of head necessary and the large volume of wash-water required. It would appear from Mr. Weston's theoretical discussion that, at times, aeration alone will not suffice to effect the precipitation of the oxidized iron, owing to the colloidal form of the ferric hydrate; nor will aeration, sedimentation, and open sand filters, when organic matter is present. It appears, therefore, that the only plan always feasible (except for water similar to that at Reading, Mass.) is a combination of aerators and pressure filters, or aerators, roughing filters, and pressure filters. In other words, the precipitation of the iron is effected ordinarily by a combination of chemical oxidation and mechanical separation induced by the adsorptive effect of the ferric hydrate previously precipitated upon the filtering medium. On this account roughing filters appear favorable as a means for lightening the load on the finishing filters, and doubtless would take the place of an aerating tower, particularly if the water be raised by an air-lift. In such cases pressure or open mechanical filters are to be considered, and not slow sand filters.

Mr. Potter. Alexander Potter, Assoc. M. Am. Soc. C. E. (by letter).—The author has made an exhaustive and valuable research. The presence of iron and manganese in ground-waters is well established. The purification of such waters is an important subject, and has been treated fully in this paper. Although the author states that the paper "does not concern itself with methods of water-softening or alkali reduction, but with the theory and practice of the iron-removal processes in use in Europe and America," still there are so many waters containing iron and manganese, where these constituents form the simplest and least refractory ones to be treated, that the discussion might well include processes which will soften the water and remove, as well, other impurities far more serious than iron and manganese.

Along the Southern Atlantic and also along the Gulf Coast, well-waters containing iron are for the most part also hard. This is also true of many other well-waters throughout the country. In Western Pennsylvania, both surface and well-waters contain iron. Some of the sources of this iron are mine drainage, washings from coke ovens, and refuse from mills. Certain of these waters contain more iron than many of the supplies specifically mentioned by the author. For instance, the supply at McKeesport, Pa., has at times as much as 25 parts per million of iron, whereas the supply at Worms contains 0.5; Reading, Mass. (November, 1891, highest), 13.3; Berlin, 1.2; New Orleans, 3.2; Provincetown, Mass., 3.6.

There is no doubt whatever that for certain classes of water, including those which are naturally soft, the method described and advocated by the author will prove the cheapest and best; but a comparison of the figures derived from the diagram, Fig. 1, showing

the cost of the filtration of river waters and the deferrization of Mr. Potter. ground-waters, with the actual cost of water-softening plants which have come within the writer's observation, would indicate that water-softening processes can be installed oftentimes with very little more expense than for a plant constructed simply for the deferrization of the water; with the added advantage of giving a greater return upon the investment and producing a water which is much more satisfactory for all purposes. Where the water is hard and ferruginous, there is no difficulty about the removal of the iron by aeration or coagulation without attempting softening, and the supply is thus improved, but the relatively small additional cost of providing for the softening of the water warrants the consideration of a complete softening plant, rather than one simply for deferrization.

The water of the Youghiogheny River perhaps contains more iron than that of any other river in that part of the country, and it is present in solution as sulphates; but the amount of iron in the river was not as great as that in the well-water which, until the commencement of the operations of the softening plant, formed a part of the supply for McKeesport. The sulphate hardness, both iron and calcium, was so great at times as to warrant the abandonment of the ground-water supply altogether, especially as the geological formation in the vicinity of the water-supply station indicated possibilities of pollution from surface sources.

The presence of the salts of iron in such waters as the Youghiogheny is of great benefit in the softening process, for when acted upon either by carbonate of soda or hydrate of lime (the two constituents most generally used in water-softening processes), the hydrate of iron is precipitated. Because of its flocculent nature, this and the hydrate of aluminum, make the best coagulating agents known. On several occasions during the testing of the McKeesport softening plant, the deficiency of carbonate of soda and hydrate of lime, during the period when the iron and aluminum content in the water was high, gave rise to a colloidal condition of the iron and aluminum hydrates so that they could not be precipitated, no matter how long the period of settlement, and they passed through the mechanical filters unchanged. These iron and aluminum salts have also the well-known action of rendering the calcium carbonate (left in the water as a result of treatment) less soluble.

An added reason why water-softening should receive more general consideration in the discussion of ground-water supplies, is the value of the settling tanks in the removal of bacteria. Analyses at McKeesport show that the settling tank is performing the bulk of the work. A test made on January 25th, 1909, gave 13 000 bacteria per cu. cm. in the raw water, and in the filtered water 50 per cu. cm. Of the total amount, 99% was removed in the settling tank. On January 29th,

Mr. Potter. 1909, a test showed that with a bacterial count of 5 000 per cu. cm. in the raw water, the settling basin effluent contained 90 per cu. cm., while the filtered water was sterile. This gives an efficiency of 98.2% for the settling tank and 100% for the entire plant.

These are representative tests. The plant has been in operation for nearly four months, having been started before its full completion because of the terrible condition of the raw water in October, 1908.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM BEVERLY CHASE, M. Am. Soc. C. E.*

DIED OCTOBER 26TH, 1908.

William Beverly Chase, the son of Levi W. and Harriet Vining Chase, was born in Marengo, Morrow County, Ohio, on November 21st, 1852. His family was of Colonial ancestry, his great-grandfather, Beverly Chase, having served as a New York Militiaman in the War of the Revolution.

His father was an architect and master builder of the old school, and, in his early years, the son lived in the atmosphere of the builder and the artist and became acquainted with the use of the draftsman's tools.

His early education was secured in the high schools provided in the Central Ohio towns of that period, and has been described as "consisting mainly in digging out things for himself," and, to his praise be it said, that during his after life he made most successful use of the ability thus acquired.

In early manhood he removed with his parents to Southwestern Minnesota, where for five years he was engaged in surveys and in acquiring practical experience in the design and construction of railroad bridges.

In 1877 Mr. Chase removed to Oregon, and for more than thirty years was identified with the growth and development of that State. For the first three years he was occupied with map work, and was in the employ of local bridge builders, but, in 1880, his opportunity came with the construction of the Northern Pacific Railroad, surveys for which were then in progress. Beginning as a Topographer for a field party, he was soon at Headquarters in Portland preparing designs and plans for the large number of structures required for the Western Divisions of the railway, then under the charge of V. G. Bogue, M. Am. Soc. C. E.

From 1884 to 1885, he was Engineer of Bridges for the Oregon Pacific Railroad, a line crossing from the Coast through the Western portion of the State, of which the late Isaac W. Smith, M. Am. Soc. C. E., was Chief Engineer. From 1885 to 1890 he was engaged in hydraulic and sanitary surveys and work and in bridge construction for some of the towns of the State, among which were Corvallis and Eugene.

In 1891 Mr. Chase made surveys and designs for a sewer system

* Memoir prepared by D. D. Clarke, M. Am. Soc. C. E.

for East Portland, and, from 1891 to 1895, he was Engineer of Bridges for the Portland Bridge Commission, during which time he constructed the Burnside Street Bridge, crossing the Willamette River, costing $300 000, and a steam ferry-boat for North Portland. From 1894 to 1896 he was engaged in general practice, while from 1896 to 1902 he was in the service of the City, first as Superintendent of Streets and later as City Engineer, retaining the latter position until, by a change in political parties, the wing with which he had allied himself suffered defeat.

During all these years and until his last illness, he continued to act as Consulting Engineer for the County Commissioners, who under the law are charged with the duty of maintaining all the bridges and ferries crossing the Willamette within the City limits. After the retirement from the service of the City, he was engaged in making surveys and designs for various towns for water supply, street pavements, etc., the Towns of Astoria, Corvallis, McMinnville, Rainier, and Tillamook, Oregon, and North Yakima, Washington, being among the places which he served acceptably.

It was while engaged in making an examination of a water-supply project for the Town of McMinnville, in July, 1908, that Mr. Chase suffered a severe paralytic stroke from which he never recovered, being confined to his bed from that time until his death, which occurred October 26th, 1908, at the Good Samaritan Hospital, Portland.

Mr. Chase was known as a genial gentleman and a man of recognized ability and worth. He was a Christian, from early life, and although not demonstrative he was ever known as a loyal supporter of all things that make for that "righteousness which exalteth the nation." He was long connected with the Centenary Methodist Episcopal Church of Portland, Oregon, and by his loyalty and steadfastness was largely instrumental in sustaining it during a critical period of its history. His zeal in the service of his church was his by inheritance from a godly ancestry.

In 1884 Mr. Chase was married to Miss Georgia Parker of Astoria, Oregon. The death of his wife in 1894, leaving him with their family of three little daughters, and the care of his aged father, greatly modified Mr. Chase's purposes and efforts in his profession, obliging him to put aside congenial opportunities offering exercise of his energies in wider fields, but through all his life he was an inspiring example of what may be accomplished alone, following a natural bent, supplemented by faithful application and courage.

Mr. Chase is survived by two daughters, Misses Marion and Jessie Chase, of Portland, Oregon, and by a brother, the Rev. Charles E. Chase, of San Francisco, and a sister, Mrs. Lucia C. Bell, of Fruitvale, California.

Mr. Chase was elected a Member of the American Society of Civil Engineers on September 6th, 1899.

William P. [illegible]

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XXXV—No. 3

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ENGINEERS
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1852

March 1909

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(INSTITUTED 1852)

VOL. XXXV—No. 3.

MARCH, 1909

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1909

American Society of Civil Engineers

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER......................5913 Columbus.
CABLE ADDRESS........................"Ceas, New York."

Vol. XXXV. MARCH, 1909. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

February 17th, 1909.—The meeting was called to order at 8.30 P. M.; George W. Tillson, Director Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 102 members and 6 guests.

A paper by R. P. Bolton, M. Am. Soc. C. E., entitled "The Operation of Passenger Elevators," was presented by the author and discussed orally by Messrs. R. B. Stanton, W. J. Boucher, F. Lavis, Noah Cummings, and the author.

Adjourned.

March 3d, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates, in the chair; Chas. Warren Hunt, Secretary; and present, also, 252 members and 50 guests.

The minutes of the Annual Meeting, January 20th, and of the meeting of February 3d, 1909, were approved as printed in the *Proceedings* for February, 1909.

The President appointed Messrs. T. Kennard Thomson, Frank E. Winsor, and A. S. Nye as Tellers to canvass the ballot on the following proposed amendment to the Constitution:

Amend Article III as follows:

Section 1.—Amend the last paragraph to read as follows:

> "All members other than Honorary Members and Juniors shall be admitted to the Society only by vote of the Corporate Members, as hereinafter specified."

Section 3.—Amend the last sentence of the first paragraph to read as follows:

> "Not less than twenty days after the issue of such list, the Board of Direction shall consider these applications, together with any information in regard to the applicants that may have been received; may make further inquiries, if deemed expedient; shall classify the applicant, with his consent, and on applications for admission may direct a ballot."

Amend the last paragraph to read as follows:

> "The Board shall have the power to elect persons to the grade of Junior, and to transfer persons from any grade except that of Junior to a higher grade of membership, and shall notify the Membership of its action."

Section 4.—Amend Section 4 to read as follows:

> "The ballots shall be letter-ballots, in a form to be prescribed by the Board of Direction. They shall be mailed to each Corporate Member whose address is known, and shall state the date on which the ballot is to be canvassed, which shall be not less than twenty days after the issue of the ballot. **Negative votes equal to one per cent., or to the whole number nearest to one per cent., of the total corporate membership at the time of voting shall exclude from election.**
>
> "A rejected applicant may renew his application for membership at any time after the expiration of one year from the date of the ballot rejecting his previous application."

The Tellers reported as follows:

On the adoption of the amendment to Article III, relating to the Method of Election of Members:

Total number of votes received		1 016
Not entitled to vote		3
Counted		**1 013**
No	762	
Yes	247	
Defective	4	
		1 013

An affirmative vote of two-thirds of all ballots cast being necessary for the adoption of an amendment to the Constitution, the President declared the amendment lost.

A paper by Messrs. Ernest R. Matthews and James Watson, entitled "The Action of Frost on Cement and Cement Mortar, Together With Other Experiments on These Materials," was presented by title and discussed orally by Messrs. J. L. Davis, R. B. Stanton, E. W. Stern, and John C. Wait; and the Secretary read communications on the subject from Messrs. William M. Venable and Herbert W. Hatton.

A paper by E. P. Goodrich, M. Am. Soc. C. E., entitled "The Bonding of New to Old Concrete," was presented by the author and discussed orally by Messrs. E. J. Fort, J. L. Davis, Myron H. Lewis, E. W. Stern, and the author. A communication on the subject from H. R. Burroughs, Jun. Am. Soc. C. E., was presented by the Secretary.

The Secretary announced the election of the following candidates by the Board of Direction on March 2d, 1909:

As Members.

Thomas Throp Allard, Havana, Cuba.
William Wallace Atterbury, Philadelphia, Pa.
Herschel Albert Benedict, Albany, N. Y.
Charles Slauson Boardman, Buffalo, N. Y.
Edward Webster Crellin, Des Moines, Iowa.
Louis Curtis Kelsey, Salt Lake City, Utah.
Guy McMurtry, Yuba City, Cal.
James Augustus Nelson, Pittsburg, Pa.
Samuel Henry Pitcher, Worcester, Mass.
Erdis Geroska Robinson, Columbus, Ohio.
Gilman Walter Smith, Chicago, Ill.

As Associate Members.

Robert Fleming Arnott, New York City.
John Charles Blaylock, Chicago, Ill.
Paul Bruce Breneman, State College, Pa.
Alexander Chisholm Copland, Richmond, Va.
Charles Wells Eddy, Thomaston, Conn.
Oscar Llewellyn Grover, Richmond, Va.
Benjamin Alexander Hodgdon, New York City.
Frank Rhymal Judd, Chicago, Ill.
Harlow Barton Kirkpatrick, Manila, Philippine Islands.
Morton Macartney, Spokane, Wash.
John Joseph Murphy, Yonkers, N. Y.
Thomas Jett Powell, Washington, D. C.
Eugene Jesse Rights, New York City.

As Associate.

Edward Michael Graves, Cleveland, Ohio.

As Juniors.

Harris Arkush Appel, Denver, Colo.
Franklin Babcock, Corregidor Island, Philippine Islands.
Harry Everett Barnes, New York City.
Fred Eberspacher, Vicksburg, Miss.
Harold Frederick Forsyth, Lone Tree, Wash.
Leon Cohen Heilbronner, Schenectady, N. Y.
Grover Cleveland Pruett, Miles City, Mont.
Paul Francis Rossell, Juana Diaz, Porto Rico.
Charles Willett Spooner, Ann Arbor, Mich.
Edwin Delevan Tillson, New York City.
Benjamin Franklin Vandervoort, New York City.
Louis Wachtel, Albany, N. Y.

The Secretary announced the transfer of the following candidates by the Board of Direction on March 2d, 1909:

From Associate Member to Member.

Clarence Edson Alderman, Boston, Mass.
Harry Franklin Bascom, Allentown, Pa.
Henry Baum Brewster, Syracuse, N. Y.
Charles Albert McKenney, Washington, D. C.
Eugene Everett Pettee, Boston, Mass.
Francis Lansing Pruyn, New York City.
Walter Evans Spear, Brooklyn, N. Y.
William MacIntire White, New York City.

From Junior to Associate Member.

Francis Fielding Longley, Washington, D. C.
Robert Faulkner Moss, Iloilo, Philippine Islands.
Alfred Marshall Wyman, East Orange N. J.

The Secretary announced the transfer of the following candidate by the Board of Direction on February 2d, 1909:

From Associate to Associate Member.

Richard Roswell Lyman, Salt Lake City, Utah.

The Secretary announced the election of the following candidate by the Board of Direction on September 1st, 1908:

As Junior.

Julius Lilien Jacobs, Houston, Tex.

The Secretary announced the following death:

Francis Smith Burrowes, elected Member June 6th, 1888; died February 15th, 1909.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, April 7th, 1909.—8.30 P. M.—A paper on "The Sixth Street Viaduct of Kansas City," by E. E. Howard, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper was printed in *Proceedings* for February, 1909.

Wednesday, April 21st, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "The Maximum Weights of Slow Freight Trains," by C. S. Bissell, M. Am. Soc. C. E.; and "Sampittic Surfacing," by W. W. Crosby, M. Am. Soc. C. E.

These papers were printed in *Proceedings* for February, 1909.

Wednesday, May 5th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "A System of Cost Keeping," by Myron S. Falk, Assoc. M. Am. Soc. C. E.; and "The Design of Elevated Tanks and Stand-Pipes," by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E..

These papers are printed in this number of *Proceedings.*

Wednesday, May 19th, 1909.—8.30 P. M.—A paper entitled "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," by C. E. Grunsky, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from July 6th to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

H. W. Hayes,

A. W. Dean, S. E. Tinkham,

H. D. Woods, J. F. Stevens,

George A. Kimball, J. W. Ellis.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) will be issued in about ten days. There will be three additional volumes issued for this year. As will be seen, this volume is as large as those recently issued semi-annually.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume for 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Club of Cleveland, 718 Caxton Building, Cleveland, Ohio.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur= und Architekten=Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur= und Architekten=Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

* *Proceedings*, Vol. XXXIII, p. 20 (January, 1907).

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association

(Abstract of Minutes of Meeting)

December 16th, 1908.—The annual meeting of the San Francisco Association of Members, Am. Soc. C. E., was held; President C. D. Marx in the chair; Franklin Riffle, Secretary; and present, also, 38 members and guests.

The minutes of the previous meeting were approved as printed in the *Proceedings* of the Association.

The Secretary presented his annual report. Since the last annual report, six meetings have been held, the average attendance being 38. There have been 8 additions to the membership, and 5 names have been dropped, making the present membership 102, a net gain of 3.

The Treasurer reported that the receipts for the year had been $1 505.49, and the disbursements $549.65, leaving a balance of $955.84.

The Secretary read a letter from Charles Warren Hunt, Secretary of the American Society of Civil Engineers, conveying to the Association (as suggested by the Committee to Recommend the Award of Prizes) the appreciation of the Board of Direction of the Society for the work done by the Association in preparing papers pertaining to the "Effects of the San Francisco Earthquake of April, 1906."

The Secretary was instructed to convey to the Secretary of the Society the thanks of the Association for the words of commendation contained in his letter.

The following officers were elected:

President, LUTHER WAGONER.
Vice-Presidents, J. D. GALLOWAY,
C. B. WING.
Secretary, E. T. THURSTON, JR.
Treasurer, P. E. HARROUN.

Mr. L. J. Mensch gave an interesting talk concerning the manufacture and driving of 190 reinforced concrete piles for the power station of the Pacific Gas and Electric Company, in Oakland, Cal.

Adjourned.

Colorado Association

The Colorado Association of Members of the American Society of Civil Engineers was organized at a meeting held in Denver, Colo., in December, 1908. A constitution, which has subsequently been approved

by the Board of Direction of the Society, was adopted February 13th, 1909, and the following officers were elected:

President, Herbert S. Crocker.
Vice-President, George G. Anderson.
Secretary-Treasurer, Henry J. Burt.

Meetings are held on the second Saturday of each month except July and August.

THE JOHN FRITZ MEDAL

The John Fritz Medal for 1909 has been awarded to Mr. Charles T. Porter for his work in "advancing the knowledge of steam engineering and in improvements in engine construction."

It has been decided that the presentation of this medal shall be made at a public meeting to be held in the auditorium of the Engineering Societies Building, 29 West 39th Street, on the evening of Tuesday, April 13th, 1909.

Addresses will be made on the following subjects:

"The Debt of Modern Industrial Civilization to the Steam Engine,"
"The Debt of the Steel Rolling Mill to the High-Speed Engine,"
"The Debt of the Electrical Generator to the High-Speed Engine,"
"The Debt of the Modern Steam Engine to Charles T. Porter."

All members of this Society are invited to attend this meeting.

ACCESSIONS TO THE LIBRARY

(From February 10th to March 9th, 1909)

DONATIONS.*

THEORY AND DESIGN OF REINFORCED CONCRETE ARCHES.

A Treatise for Engineers and Technical Students. By Arvid Reuterdahl. Cloth, 9 x 6 in., illus., 3 + 126 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $2.00 net.

The claim of the author of this work, as set forth in his preface, is that there has been no previous treatise on the analysis of the elastic arch which was not either so mathematically abstruse, or left so much to the reader to demonstrate for himself, as to be of little use to the practical engineer or technical student whose mathematical training has not been of an exceptional order. He further claims that every possible principle involved in the graphical treatment of analysis is explained thoroughly in the theoretical portion of his work, and that there are no missing steps in the necessary mathematical analysis of the theory, as set forth in the treatise. The Contents are: Theory of the Elastic Arch; Design of a Reinforced Concrete Arch; Calculations of Fiber Stresses; Index.

ASPHALTS: THEIR SOURCES AND UTILIZATIONS.

Asphalts for Dustless Roads; Recent Improvements in Asphalt Industries, Together with Addenda Treating on General Waterproof Construction. By T. Hugh Boorman. Cloth, 10 x 6½ in., illus., 168 pp. New York, William T. Comstock, 1908. $3.00.

The author, in his preface, states that he has endeavored to furnish a complete manual on asphalt, which shall be a reliable reference book. The book was suggested, he says, by a series of articles that appeared in *Architects' and Builders' Magazine*, for which, since the numbers containing them ran out of print, there has been a continuous call, and the request was made that they be issued in book form. The Contents are: Discovery and Early Use of Asphalt; Rock Asphalt Mastic or Asphalt Coulé; Trinidad Asphalt; Petroleum Residuum and California Malthas as a Fluxing Material; Venezuela Asphalts; Cuban Asphalts; American Bituminous Limestone; Bituminous Asphalt Sandstone Rock; Manjak and Uintaite; Late European Work; Turkish and Other Bitumens; Developments of Asphalt Industry up to 1903; Asphalts in 1908; Asphalt in Building Construction; Dustless Roads; Methods of Surfacing Roads; Asphaltic Oils, Their Classification and Properties; Application of Asphaltic Oils; Sprinkling with Asphaltic Oils; Latest Views of Engineers on Asphaltic Surfacing; Municipal Asphalt Plants; Asphalt Waterproofing; Asphalt in Roofing; Asphalt in Manufacture; Asphalt Machinery; Index.

ENGINEERS' POCKETBOOK OF REINFORCED CONCRETE.

By E. Lee Heidenreich. Roan, 7 x 4½ in., illus., 9 + 364 pp. Chicago and New York, The Myron C. Clark Publishing Co.; London, E. & F. Spon, Limited, 1909. $3.00 net.

The preface states that the author has been writing, changing, and improving this book for over eight years, and that, owing to the comparative newness of the subject and its possibilities for additions and amendments, it has been almost impossible to find a place at which it could be considered even temporarily finished. A pocketbook on the subject being much needed, however, the author here gives, more or less concisely, the result of about fifteen years of study and experience in exploitation and construction of reinforced concrete. The Contents are: Materials and Machines Used in Reinforced Concrete Construction; Design and Construction of Buildings; The Design and Construction of Bridges; Abutments and Retaining Walls; Culverts, Conduits, Sewers, Pipes, and Dams; Tanks, Reservoirs, Bins, and Grain Elevators; Chimneys, Miscellaneous Data, Cost Keeping, Estimating, Specifications, Etc.; Index.

*Unless otherwise specified, books in this list have been donated by the publisher.

GENERAL LECTURES ON ELECTRICAL ENGINEERING.

By Charles Proteus Steinmetz. Edited by Joseph Le Roy Hayden. Cloth, 9 x 6 in., illus., 284 pp. Schenectady, N. Y., Robson & Adee. $2.00 net.

This book, the publishers announce, is made up of a series of lectures, dealing with the operation of electric systems and apparatus under normal and abnormal conditions, and with their design. The author's preface states, however, that the design of apparatus has been discussed only so far as it is necessary to understand their operation and so judge of their proper field of application, and that the lectures comprise a discussion of the different methods of application of electric energy, the means and apparatus available, the different methods of carrying out the purpose, and the relative advantages and disadvantages of the different methods and apparatus which determine their choice, the treatment being essentially descriptive and not mathematical. The Lecture Headings are: General Review; General Distribution; Light and Power Distribution; Load Factor and Cost of Power; Long Distance Transmission; Higher Harmonics of the Generator Wave; High Frequency Oscillations and Surges; Generation; Hunting of Synchronous Machines; Regulation and Control; Lightning Protection; Electric Railway, Train Characteristics; Electric Railway, Motor Characteristics; Alternating Current Railway Motor; Electro-chemistry; The Incandescent Lamp; Arc Lighting; Appendix I, Light and Illumination; Appendix II, Lightning and Lightning Protection.

AUDEL'S GAS ENGINE MANUAL.

A Practical Treatise Relating to the Theory and Management of Gas, Gasoline, and Oil Engines, Including Chapters on Producer Gas Plants, Marine Motors, and Automobile Engines. Cloth, $8\frac{1}{2}$ x $5\frac{1}{2}$ in., illus., 24 + 469 pp. New York, Theo. Audel & Co. $2.00.

The Contents are: Historical Development; Laws of Permanent Gases; Theoretical Working Principles; Actual Working Cycles; Graphics of the Action of Gases; Indicator Diagrams of Engine Cycles; Indicator Diagrams of Gas Engines; Fuels and Explosive Mixtures; Gas Producer Systems; Compression, Ignition and Combustion; Design and Construction; Governing and Governors; Ignition and Igniters; Installation and Operation; Four-Cycle Horizontal Engines; Four-Cycle Vertical Engines; Four-Cycle Double-Acting Engines; Two-Cycle Engines; Foreign Engines; Oil Engines; Marine Engines; Testing; Instruments Used in Testing; Nature and Use of Lubricants; Hints on Management and Suggestions for Emergencies; The Automobile; Useful Rules and Tables; Index.

PATENTS AS A FACTOR IN MANUFACTURING.

By Edwin J. Prindle. Cloth, $7\frac{3}{4}$ x 5 in., 134 pp. New York, The Engineering Magazine, 1908. $2.00.

The author of this book disclaims any purpose to make the inventor or manufacturer his own patent attorney, and the editor's preface states it to be designed especially to lay down the fundamental principles, so that they may be grasped clearly and fully enough to direct the course of the inventor, patentee, or manufacturer, in the early steps which are usually taken before the advice of counsel is secured. The Contents are: Influence of Patents in Controlling a Market; Subject, Nature, and Claim of a Patent; What Protection a Patent Affords; Of Infringements; Patenting a New Product; Patent Relations of Employer and Employee; Contests Between Rival Claimants to an Invention; Index.

PRACTICAL CALCULATION OF TRANSMISSION LINES

For Distribution of Direct and Alternating Currents by Means of Overhead, Underground, and Interior Wires for Purposes of Light, Power, and Traction. By L. W. Rosenthal. Cloth, $9\frac{1}{2}$ x 6 in., 10 + 93 pp. New York, McGraw Publishing Company, 1909. $2.00 net.

The chief mission of this book is stated by the author to be to substitute a direct solution for the trial method of calculation of transmission lines which was formerly a necessary evil. The scope of the book has been confined to methods of calculation, it is stated, and the arrangement of the formulas, tables, and text dictated solely by the needs of the rapid worker. All sections, except the last, include the effects of temperature and specific conductivity. The section relating to direct-current railways is claimed to be novel in the form of its tables, and the methods outlined in it, rapid and comprehensive; that relating to alternating-

current transmission is said to present an original method for the solution of these problems. The Chapter Headings are: Direct-Current Distribution for Light and Power; Distribution for Direct-Current Railways; Alternating-Current Transmission by Overhead Wires; Alternating-Current Transmission by Underground Cables; Interior Wires for Alternating-Current Distribution; Distribution for Single-Phase Railways.

HEAT ENERGY AND FUELS.

Pyrometry, Combustion, Analysis of Fuels, and Manufacture of Charcoal, Coke, and Fuel Gases. By Hanns v. Jüptner. Translated by Oskar Nagel. Cloth, $9\frac{1}{2}$ x 6 in., illus., 5 + 306 pp. New York, McGraw Publishing Company, 1908. $3.00 net.

The translator is authority for the statement that this volume contains a large amount of carefully tabulated data, of which a great deal is new, in convenient form for use, and that while the book is intended for use in universities and engineering schools, it is of equal value to practising engineers, since it gives, not only the fundamental principles, but also the latest experimental data and practice. The Contents are: General Remarks; Forms of Energy; The Measurement of High Temperatures (Pyrometry); Pyrometry (continued); Pyrometry (conclusion); Optical Methods of Measuring Temperatures; Combustion Heat and Its Determination; Direct Methods for Determining the Combustion Heat; Incomplete Combustion; Combustion Temperature; Fuels (In General); Wood; Fossil Solid Fuels (In General); Peat; Brown Coal (Lignite); Bituminous and Anthracite Coals; Artificial Solid Fuels; Charcoal; Peat-Coal, Coke, and Briquettes; Coking Apparatus; Liquid Fuels; Gaseous Fuels; Producer Gas; Water Gas; Dowson Gas, Blast Furnace Gas, and Regenerated Combustion Gases; Apparatus for the Production of Fuel Gases; Index.

LAW AND BUSINESS OF ENGINEERING AND CONTRACTING.

With Numerous Forms and Blanks for Practical Use. By Charles Evan Fowler, M. Am. Soc. C. E. Cloth, 9 x 6 in., 9 + 162 pp. New York, McGraw Publishing Company, 1909. $2.50 net.

The author states in his preface that this book is the elaboration of a series of lectures delivered to the engineering students of Washington University, with some additional matter in the shape of forms of contracts, specifications, and blank business forms. The Contents are: The Relation Between the Engineer and Contractor; Ordinary Forms of Contracts; Ordinary Specifications; Special Forms of Specifications; Special Forms of Contracts; Inspection of Engineering Work; Estimating Materials and the Cost of Engineering Structures; Bidding on Engineering Work; Organization of Contract Work; Essentials of Contract Law; Index.

THE OCEAN CARRIER.

A History and Analysis of the Service, and a Discussion of the Rates of Ocean Transportation. By J. Russell Smith. Cloth, $7\frac{3}{4}$ x 5 in., illus., 11 + 344 pp. New York and London, G. P. Putnam's Sons, 1908. $1.50.

The author calls this book primarily an economic study of the ocean service, and states it to be the outgrowth of the study of three questions—the development of line traffic, the combinations among carriers to control rates, and the combination of steamship lines and railways. It is written in a popular, rather than a technical style, and treats of what ships carry, where they carry it, for whom, at what rate, under what management, etc., tracing the main lines of past development and the dominant factors in the present situation. The Contents are: Part I, The Service of the Ocean Carrier: Ship Development—the Evolution of the Vehicle; The Organization of Ocean Carrying; The Leading Routes of Ocean Commerce; The Epoch of the Merchant Carrier on the Sea; Line Traffic and Its Extension; Recent Developments in Line Traffic; The Normal Type of Steamship Line Organization; The Railroad Steamship Lines on the Atlantic and Gulf Coasts of the United States; The Railroad Steamship Lines on the Pacific Coast of the United States and in Europe; The Renaissance of the Merchant Carrier—The Private Steamship Line; Line Traffic in the United States Coasting Trade. Part II, The Rates of the Ocean Carrier: Charter Freight Rates and Attempts at Their Control; Factors Affecting Ocean Line Freight Rates; Agreements Among Ocean Carriers to Control Rates; Transatlantic Freight Rates and Their Control—The Lines Between America and Great Britain; Transatlantic Freight Rates and Their Control—The Lines Between America and the Continent; The Present Situation and Future Outlook; Index.

LABORATORY EXPERIMENTS IN METALLURGY.

By Albert Sauveur and H. M. Boylston. Cloth, $10\frac{1}{4}$ x $7\frac{3}{4}$ in., illus., 73 pp. Cambridge, Mass., The Authors, 1908. $1.25. (Presented by the Authors.)

This book is made up of short notes written primarily for the use of students at Harvard University, who take the courses in General Metallurgy and in Metallurgy of Iron and Steel. The authors' intention, as indicated in the preface, is not to deal with the subject exhaustively, but to make the notes suggestive to other teachers and students in metallurgy, and even to practitioners of the art. The book is divided into two parts of thirteen experiments each, Part I dealing with experiments in general metallurgy, and Part II with experiments in metallurgy of iron and steel. There are various tables in the text such as Typical Analyses of Various Fuels, Melting Points of Metals, etc., and illustrations of apparatus. After each experiment a blank laboratory report is given to be filled out by the student.

Gifts have also been received from the following:

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Washington Southern Ry. Co. 3 pam.
Western Australia-Dept. of Mines. 3 pam.

BY PURCHASE.

Coal. By James Tonge. New York, D. Van Nostrand Co., 1907.

The Mechanical Engineering of Steam Power Plants. By F. R. Hutton. Ed. 3, Rewritten. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1909.

The Temperature-Entropy Diagram. By C. W. Berry. Ed. 2, Rev. and Enl. New York, John Wiley & Sons; London, Chapman & Hall, Limited, 1908.

Die Eisernen Brücken im Allgemeinen: Theorie der Eisernen Balkenbrücken. Handbuch der Ingenieur Wissenschaften. II. Teil, dritter Band, vierte vermehrte Auflage. Bearbeitet von J. E. Brik, Th. Landsberg und Fr. Steiner. Leipzig, Wilhelm Engelmann, 1909.

Physical Geography of the Texas Region. By R. T. Hill. Topographic Atlas of the United States. Washington, United States Geological Survey, 1900.

Steam-Boilers. By C. H. Peabody and E. F. Miller. Ed. 2, Rev. and Enl. New York, John Wiley & Sons; London, Chapman & Hall, 1908.

SUMMARY OF ACCESSIONS

(From February 10th to March 8th, 1909)

Donations (including 18 duplicates).............	166
By purchase..............................	6
Total	172

MEMBERSHIP

ADDITIONS

(February 10th to March 9th, 1909)

MEMBERS		Date of Membership
ALDERMAN, CLARENCE EDSON. Engr. and Contr. (Gascoigne, Alderman & Shattuck), 49 Federal St., Boston, Mass.....	Assoc. M. M.	Sept. 6, 1905 Mar. 2, 1909
ATTERBURY, WILLIAM WALLACE. Gen. Mgr., Penn. Lines East of Pittsburg and Erie, P. R. R., Broad St. Station, Philadelphia, Pa.		Mar. 2, 1909
BENEDICT, HERSCHEL ALBERT. 129 Western Ave., Albany, N. Y.		Mar. 2, 1909
BOARDMAN, CHARLES SLAUSON. 798 Ellicott Sq. Bldg., Buffalo, N. Y.		Mar. 2, 1909
BREWSTER, HENRY BAUM. Div. Engr., Middle Div., Dept. of State Engr., Syracuse, N. Y.	Assoc. M. M.	Mar. 6, 1907 Mar. 2, 1909
BROWN, EARL IVAN. Capt., Corps of Engrs., U. S. A., P. O. Drawer 813, Wilmington, N. C.		Feb. 2, 1909
COANE, JOHN MONTGOMERY. 70 Queen St., Melbourne, Victoria, Australia		Nov. 4, 1908
ENDEMANN, HERMAN KARL. Asst. Engr. in Charge, Topographical Bureau, Borough of Queens, 253 Jackson Ave., Long Island City, N. Y.	Assoc. M. M.	Mar. 6, 1901 Jan. 5, 1909
FOX, JOHN ANGELL. Special Director, National Rivers and Harbors Congress, 204 E. Front St., Cincinnati, Ohio.		Feb. 2, 1909
GRAVES, WALTER HAYDEN. 316 Wells Fargo Bldg., Portland, Ore.		Feb. 2, 1909
McKENNEY, CHARLES ALBERT. Cons. Engr., Hibbs Bldg., Washington, D. C.	Jun. Assoc. M. M.	Dec. 4, 1894 Dec. 1, 1897 Mar. 2, 1909
MURPHY, DANIEL WILLIAM. Project Engr., U. S. Reclamation Service, Klamath Falls, Ore.		Feb. 2, 1909
PETTEE, EUGENE EVERETT. (J. R. Worcester & Co.), 79 Milk St., Boston, Mass.	Assoc. M. M.	Sept. 3, 1902 Mar. 2, 1909
PRUYN, FRANCIS LANSING. Cons. Engr., 90 West Broadway, New York City.	Jun. Assoc. M. M.	Dec. 1, 1896 June 7, 1899 Mar. 2, 1909
RANDLE, GEORGE NELSON. City Engr., City Hall, Sacramento, Cal.		Feb. 2, 1909
SCHUBERT, FREDERICK CELESTINE. U. S. Asst. Engr., Custom House, Portland, Ore.	Assoc. M. M.	June 5, 1907 Feb. 2, 1909
SELANDER, JOHN EINAR. Care, Res. Engr., Lagos Ry., Northern Extension, Jebba, Lagos, Southern Nigeria		Nov. 4, 1908

MEMBERS (*Continued*)

		Date of Membership
SPEAR, WALTER EVANS. Chf. Engr., Dept. of Water Supply, Gas and Electricity, Borough of Brooklyn, Municipal Bldg., Brooklyn, N. Y.	Assoc. M. M.	Feb. 3, 1904 Mar. 2, 1909
TURNER, ORVILLE HICKMAN BROWNING. Chf. Engr., St. L., R. M. & P. Ry. Co., Raton, N. Mex.		Feb. 2, 1909
VEUVE, ERLE LEROY. Chf. Engr., Ontario & San Antonio Heights R. R., 697 Pacific Electric Bldg., Los Angeles, Cal.	Jun. M.	Sept. 3, 1901 Feb. 2, 1909
WALKER, MERIWETHER LEWIS. Maj., Corps of Engrs., U. S. A., P. O. Box 1027, Memphis, Tenn.		Feb. 2, 1909

ASSOCIATE MEMBERS

ARNOTT, ROBERT FLEMING. Cons. Engr., 95 Liberty St., New York City		Mar. 2, 1909
BROWN, MARSHALL WRIGHT. Middleport, N. Y.		Nov. 4, 1908
COLE, OSMAN FRED. 1307 Security Bldg., Chicago, Ill.		Sept. 2, 1908
COPLAND, ALEXANDER CHISHOLM. Chf. Draftsman, Constr. Dept., Chesapeake & Ohio Ry., Richmond, Va.		Mar. 2, 1909
CUNNINGHAM, JOHN GEORGE LAWRENCE. 638 Hague Ave., St. Anthony Hill, St. Paul, Minn.		Jan. 5, 1909
DE LA MATER, STEPHEN TRUESDELL. Greenwood Inn, Evanston, Ill.		Nov. 4, 1908
EDDY, CHARLES WELLS. Thomaston, Conn.		Mar. 2, 1909
GRANT, KENNETH CROTHERS. Asst. Engr., Water Supply Comm. of Pennsylvania, Harrisburg, Pa.		Feb. 2, 1909
GROVER, OSCAR LLEWELLYN. Asst. Engr., C. & O. Ry., 8th and Main Sts., Richmond, Va.		Mar. 2, 1909
HODGDON, BENJAMIN ALEXANDER. 180 N. 12th St., Newark, N. J.		Mar. 2, 1909
KRELLWITZ, DIEDRICH WILLIAM. 423 Thirteenth St., West New York, N. J.	Jun. Assoc. M.	Oct. 31, 1905 Feb. 2, 1909
LYMAN, RICHARD ROSWELL. Cons. Engr.; Prof. of Civ. Eng., 34 Engineering Bldg., Univ. of Utah, Salt Lake City, Utah	Assoc. Assoc. M.	May 4, 1904 Feb. 2, 1909
PACE, FULTON. Asst. Irrig. Engr., Guayama, Porto Rico	Jun. Assoc. M.	Nov. 5, 1907 Feb. 2, 1909
WALTER, GEORGE SHIRLEY. 806½ Marquette Bldg., Chicago, Ill.		Jan. 5, 1909
WENIGE, ARTHUR EMIL. Asst. Engr., Dept. of Water Supply, Borough of Richmond, Borough Hall, New Brighton, N. Y.	Jun. Assoc. M.	May 6, 1902 Oct. 7, 1908
WYMAN, ALFRED MARSHALL. Asst. Engr., Public Service Comm., 1st Dist., 154 Nassau St., New York City (Res., 109 North 16th St., East Orange, N. J.)	Jun. Assoc. M.	Jan. 2, 1906 Mar. 2, 1909

ASSOCIATE MEMBERS (*Continued*)	Date of Membership.
YOUNG, CHARLES NEWTON. 2086 Bush St., San Francisco, Cal.	Feb. 2, 1909

ASSOCIATES

GRAVES, EDWARD MICHAEL. 17 Commercial Bank Bldg., Cleveland, Ohio	Mar. 2, 1909
HERRON, GEORGE MERRICK. Box 1054, Palo Alto, Cal. { Jun.	April 2, 1907
{ Assoc.	Feb. 2, 1909

JUNIORS

HEILBRONNER, LEON COHEN. 238 Union St., Schenectady, N. Y.	Mar. 2, 1909
JACOBS, JULIUS LILIEN. Care, James Stewart & Co., First National Bank Bldg., Houston, Tex.	Sept. 1, 1908
SCHMID, ROBERT JOHN. Office of City Engr., Spokane, Wash.	Feb. 2, 1909
SEABURY, ARTHUR GRAY. 25 Grotto Ave., Providence, R. I.	Sept. 1, 1908
SIMPSON, CHARLES RANDOLPH. Civ. and Hydr. Engr., Warren, Pa.	Feb. 2, 1909
TYLER, WILLIAM ROGERS. 36 West 93d St., New York City.	Feb. 2, 1909
VANDERVOORT, BENJAMIN FRANKLIN. 899 Eagle Ave., New York City	Mar. 2, 1909
WACHTEL, LOUIS. Care, State Highway Comm., Lyon Blk., Albany, N. Y.	Mar. 2, 1909
WRIGHT, THOMAS TEMPLE. Box 333, Greenville, Miss.	Sept. 1, 1908

CHANGES OF ADDRESS

MEMBERS

ALLEN, WILLIAM ANDREW. Supt. of Constr., Am. Smelting & Refining Co., 165 Broadway, New York City.

BAKER, HOLLAND WILLIAMS. U. S. Engr. Office, Vicksburg, Miss.

BAYLISS, JOHN YANCEY. Prin. Asst. Engr., Madeira-Mamoré Ry., Caixa 304, Manãos, Brazil.

BRACE, JAMES HENRY. Res. Engr., P. T. & T. R. R., 345 East 33d St., New York City.

BROWN, ROBERT CALVIN. Evergreen Ave., Plainfield, N. J.

BURKE, JOHN THOMAS. Room 2, I. O. O. F. Bldg. (Res., 317 E. Jeff. St.), Boise, Idaho.

CLARK, CHARLES HENRY. Engr., Rochester Ry. Lines, Rochester, N. Y.

DOSE, HENRY FREDERICK. 32 West 40th St., New York City.

FULTON, JOHN ADDISON. 928 South Hope St., Los Angeles, Cal.

GERIG, WILLIAM. 2 North Main St., Memphis, Tenn.

HALL, JOHN LINCOLN. Care, Purdy & Henderson, 45 East 17th St., New York City.

MEMBERS (*Continued*)

HAWXHURST, ROBERT, JR. Gen. Mgr., Poderosa Min. Co., Ltd., Casilla 855, Antofagasta, Chili (also Cons. Engr., 21 Broad St. House, New Broad St., London, England).

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RESIGNATION

ASSOCIATE MEMBER	Date of Resignation
KINSEY, FRANK WILMARTH	Mar. 2, 1909

DEATH

BURROWES, FRANCIS SMITH. Elected Member, June 6th, 1888; died February 15th, 1909.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(February 9th to March 8th, 1909.)

NOTE.—*This list is published for the purpose of placing before the members of the Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(1) *Journal*, Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(70) *Engineering Review*, New York City, 10c.
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(79) *Forscherarbeiten*, Vienna, Austria.
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(82) *Dinglers Polytechnisches Journal*, Berlin, Germany.
(83) *Progressive Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
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(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
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(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(94) *The Boiler Maker*, New York City, 10c.
(95) *International Marine Engineering*, New York City, 20c.

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The Electrification of Railways: A General Comparison of Systems.* F. W. Carter. (Paper read before the Rugby Eng. Soc.) **(47)** Feb. 26.
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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XXXV. MARCH, 1909. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.

By C. E. Grunsky, M. Am. Soc. C. E.

To be Presented May 19th, 1909.

This paper ought to have been written and presented to the Engineering Profession long ago. It is now offered in order that suitable record may be made of the steps taken by San Francisco to better the primitive conditions which have been maintained there too long, and to call attention to a novel solution of the storm-water flow problem which is an outgrowth of the study of San Francisco conditions.

The writer was a member of the 'Board of Engineers' of 1892-93, generally referred to as the Sewerage Commission; he was one of the two 'Engineers in Charge to devise and provide a sewer system for San Francisco,' of 1893; he was in 1899 the 'Engineer in Charge,' with two associates, of the design of the sewerage system, which, with minor modifications, is now being carried out, and he was for more than 4 years, 1900-04, City Engineer of San Francisco, in which capacity he revised and re-submitted the plans of the sewer system, recast the

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

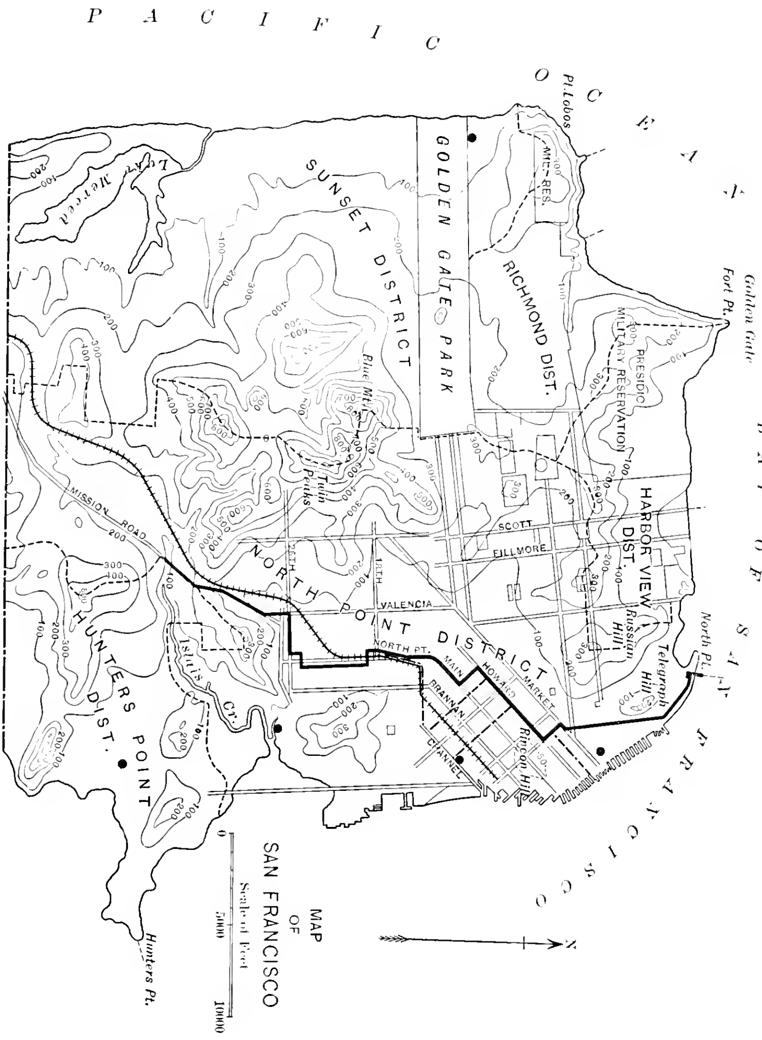

MAP
OF
SAN FRANCISCO
Scale of Feet
0
5000
10000
PACIFIC OCEAN
BAY OF SAN FRANCISCO
Golden Gate
Fort Pt.
Pt. Lobos
MTL. RES.
GOLDEN GATE PARK
SUNSET DISTRICT
RICHMOND DIST.
PRESIDIO
MILITARY RESERVATION
HARBOR VIEW
DIST.
Russian Hill
Telegraph Hill
North Pt.
NORTH POINT DISTRICT
HUNTERS POINT
DIST.
Hunters Pt.
Lake Merced
Blue Mt.
Twin Peaks
Islais Cr.
Rincon Hill
MISSION ROAD
SCOTT
FILLMORE
VALENCIA
12TH
NORTH PT.
MAIN
HOWARD
MARKET
BRANNAN
CHANNEL

FIG. 1.

cost estimate thereof, and, as funds permitted, directed the construction of the first elements of the new system. This statement is offered in explanation of the free use which has been made in this paper of the reports on the sewerage problem of San Francisco, prepared or participated in by the writer from time to time.

Topographic Features.

San Francisco covers the entire northerly portion of a peninsula in which a spur of the Coast Range, coming up from the south between San Francisco Bay and the Pacific, terminates. The city is irregularly square in outline. On the west is the Pacific Ocean; on the north, the ocean, the Golden Gate, and the bay; on the east, the bay; and on the south, Marin County, from which San Francisco is separated by an east and west line extending from the bay to the ocean.

The area of the city is about 30 000 acres, of which about one-third is built-up territory. The built-up section covers the north-eastern quadrant of the city, and extends, in long narrow strips, westward, in the northern half of the city, to the ocean, and southerly along such principal lines of travel as the San Bruno and Mission Roads to the County line. Within recent years the growth of the section on the ocean slope, particularly the region just south of Golden Gate Park, has been rapid.

The highest point in the city is practically in the city's geographical center. Here are the Twin Peaks which rise to an altitude of a little more than 900 ft. A short distance to the northwest of Twin Peaks is Blue Mountain, with a summit at practically the same height.

The ridge which divides the city into two slopes, one easterly toward the bay, the other westerly toward the ocean, enters the city from the south, a little west of a median line, and holds a fairly direct northerly course to a termination at Fort Point. The lowest points on this ridge are more than 200 ft. high. Before the recent completion of the bay shore cut-off, by the Southern Pacific Company, its main line of railroad into the city crossed this ridge from east to west just south of San Francisco, and within the city crossed back over it from west to east to reach the east central portion of the city. Parts of Golden Gate Park and the Richmond District lie upon the flat top of this ridge northward from the Twin Peaks group of hills.

The principal spurs within the city, from this peninsula backbone, which subdivide the eastern or bay slope, are, in their order from south to north: Hunter's Point ridge, terminating in a cape which extends far out into the waters of the bay in the southeastern portion of the city (south of this spur are two small valleys opening to the bay, of which the southernmost, Visitacion Valley, lies in part in San Mateo County); Bernal Heights, between two branches of Islais Creek; the Potrero Hills, which are barely connected with the main ridge by a slight swell in the ground; and the northern ridge, terminating in Russian and Telegraph Hills on the northeastern bay front of the city.

The western water-shed or ocean slope is subdivided, by the flat-topped Ocean House ridge and the Point Lobos spur, into three drainage basins, of which the northernmost is drained by Lobos Creek; the second embraces a region of drifting sand dunes extending well up on the western slope of the central group of hills; and the third is the northern part of the water-shed of Merced Lake.

Some of the hills near Point Lobos are more than 300 ft. high; the Fort Point ridge, within the Presidio, has a height of more than 400 ft.; the Presidio Heights, near where the northern ridge leaves the main ridge, have greatest elevations of 300 to 380 ft.; Lafayette Square rises to a height of 380 ft.; Clay Street Hill is 360 ft. high; Russian Hill, 370 ft., and Telegraph Hill, more than 290 ft. The lowest point on the spur which extends northwesterly from Russian Hill and terminates at Black Point is about 90 ft. high. On the Potrero Hills are points nearly 340 ft. high, and on the Hunter's Point ridge several hills rise to more than 250 ft. Railroad Avenue crosses this ridge at an elevation of 60 ft., and San Bruno Road at the same elevation. The spur which extends eastward from the main ridge south of the Hunter's Point ridge near the south line of San Francisco, leaves the main ridge with an elevation of 550 ft. This ridge separates Bay View Valley from Visitacion Valley. It is crossed by the San Bruno Road at an elevation of 140 ft., and terminates with a hill the top of which is 400 ft. high.

Bernal Heights rise to more than 490 ft.; Buena Vista Park is nearly 520, and Lone Mountain nearly 480 ft. high.

Between the several ridges or spurs of hills just mentioned lie valleys or broad flat slopes opening out upon the bay or ocean. The

ocean slope water-shed south of the Point Lobos ridge has no well-defined line of drainage. Irregular sand dunes, formed of sand blown eastward from the ocean beach, form a broad broken slope to the ocean. Some of the drifting sand has been carried over the main peninsula ridge just north of the central group of hills, and, separating into two parts, at Lone Mountain, has found lodgment in several broad belts extending eastward and southeastward across the peninsula to Mission Bay. In smaller quantity, some sand has also been carried across the peninsula south of Twin Peaks.

Sand has also been blown inland from the bay beach lying on the northern front of the city to the westward of Black Point. The great sand dunes, now only seen in small remnants, that were formerly at and near Lobos Park and some distance up on the slopes to the east and north, had their origin on this beach.

Where sand has thus drifted over portions of the site of the city it covers clays, shales, and serpentines. To some extent it encroached upon the waters of the bay on the eastern waterfront adding body to the marginal strips of swamp land and mud. Where the natural surface of the city is not covered by sand a clay soil predominates. On the hills a shaly or serpentine rock is frequently cut into when trenching for sewers, gas, or water pipes.

The northern and eastern shore of the city in its original condition was deeply indented by arms of the bay. These have in part been filled in. The most notable of these filled-in areas is the former Yerba Buena Bay, which gave the original name to San Francisco. This extended as far west as Montgomery Street and from the base of Telegraph Hill at the north to the Rincon Hill region on the south. Mission Bay, too, has been cut off and filled in. The extent of this filling will be appreciated when it is stated that here, over an area of about 150 acres, all official street grades are at City Base (6.7 ft. above ordinary high tide).

Islais Creek, south of the Potrero Hills, opens to the bay through broad marsh lands and mud flats, 500 acres of which are within the city limits. It is in this region that, fully two miles inland, official street grades as low as 1 ft. below City Base may be noted.

South of Hunter's Point an area of from 700 to 800 acres within the city limits extends out into the bay over submerged mud flats which will probably some time be filled in to about City Base.

On the northern front of the city the tidal area and bay surface that have been or that ultimately will be cut off from the bay by the established seawall line aggregate about 270 acres.

The total area of originally salt-marsh lands and bay surface within the boundaries of San Francisco is nearly 2 000 acres.

The built-up or improved sections of San Francisco fall naturally into districts of two characters: one class embracing portions of the city on sloping ground, hillsides, and hill-tops, at such elevations that they can be readily drained by gravity flow; the other embracing the low flat areas for which it is difficult or impossible to provide satisfactory sewage collection without recourse to pumping. The several districts of each class, as the foregoing description of the topography indicates, do not lie contiguous to each other. The drainage and sewerage problem is thus rendered much more complex than if there were only one instead of many drainage basins.

Water-shed Areas.—The extent of the several water-sheds, the run-off waters of which are to be cared for was in 1893 estimated as follows:

Visitacion Valley water-shed	990	acres.
Bay View water-shed	1 220	"
*Islais Creek water-shed	3 900	"
Precita Creek water-shed	1 730	"
Mission Bay water-shed	5 860	"
Yerba Buena Bay water-shed	430	"
North Beach water-shed	320	"
Washerwoman's Bay water-shed	1 030	"
Presidio water-sheds	930	"
Lobos Creek water-shed	2 120	"
Ocean Beach water-sheds	5 650	"
*Laguna de la Merced water-shed	3 350	"

In this list Precita Creek has been given independent rank. It is a tributary of Islais Creek, uniting with the latter at the margin of the salt marsh. Bernal Heights forms a prominent line of division between these two creeks.

Visitacion and Bay View Valleys.—Both Visitacion and Bay View Valleys are still sparsely settled, but the latter at least is a section in immediate need of main sewers.

* Only the area of these water-sheds within San Francisco is here noted.

Islais Creek.—The main branch of Islais Creek heads to the southward of San Francisco. It descends rapidly in its westerly course to the low marsh lands which extend far up into the valley of this creek. The creek before emerging upon these marsh lands lies for some distance in a deep gorge which takes a course through private lands. The rectangular system of laying out streets has been adhered to in some places along this creek, where, without question, there should have been streets following lines of drainage. Where lands are not yet subdivided, it is to be hoped that this matter will receive proper attention.

Islais Creek receives the run-off waters from a portion of the San Bruno Mountain, to the south of San Francisco, from the northeast slope of the Twin Peaks group of hills, and from the entire southern slope of Bernal Heights. The region which it drains, the south central portion of San Francisco, is a rapidly growing section of the city.

By statutory enactment, Islais Creek is declared to be navigable "from Franconia Landing near Bay View turnpike to its outlet into the Bay of San Francisco, and thence easterly along the southerly line of Tulare Street to the city water front on Massachusetts Street, of the width of the channel of said creek." However, Kentucky Street has been carried across the creek on a solid fill and there is still some doubt as to whether the creek, which across the marsh is but a slight depression in the mud's surface, will be opened and kept open for navigation purposes. But at the mouth of the creek a large basin should be dredged to a depth of about 40 ft. to meet in part the needs of San Francisco's growing commerce. It is foreseen that a project for such a basin will ultimately be carried out. All the Islais Creek marsh has been laid out in streets, and official grades have been established. Sometimes this was done in the most absurd fashion. Thus, on San Bruno Road and on Fifteenth Avenue, grades are as low and even lower than on Kentucky Avenue, $1\frac{1}{4}$ miles nearer the bay.

Precita Creek.—This creek drains a region which lies between the Islais Creek water-shed on the south and the Mission Bay drainage on the north. As a drainway, the creek has been replaced by Army Street sewer. The discharge of sewage is upon or through the marsh into Islais Creek, a short distance east of the eastern spur of Bernal Heights.

Mission Bay Water-shed.—The drainage basin tributary to Mission Bay is of large extent, and is for the most part densely populated. It includes the heart of the city, the entire Mission and the Western Addition. All the sewage and storm-water run-off of this region, except a relatively small portion intercepted by Brannan Street sewer, finds its way into the open waterway of Channel Street. The large Channel Street sewer, and the sewers on Seventh, Sixth, and other streets, discharge into this waterway. The discharge from these sewers represents more than half of the sewage and rain-water of the built-up portion of the city. During half the year there is no rain to dilute the sewage, and it ebbs and flows in Channel Street, creating a filthy nuisance beggaring description. There is no chance for sweeping it out into the bay because there is no circulation of water. It will continue to be offensive until the sewage is carried to some other point of outfall. But even under the contemplated arrangement to be hereinafter described the remedy will not be complete. There will still be some, though very dilute, sewage occasionally discharged into Channel Street, and this open water will remain under a disadvantage common to all long narrow inlets from a tidal bay, that of infrequent change of its water body.

The principal sewer of the Mission Bay water-shed, as now in service, is the Channel Street sewer, 11 ft. wide and 8½ ft. from invert to roof. Second in importance, if judged by size, is the Brannan Street sewer 9 ft. wide and 7 ft. high in the clear. It intersects the sewers of crossing streets; and, by reason of a connection at its head, near Tenth Street, with the Channel Street sewer, is supposed to act in a measure as a relief outlet for it. The grade of the Brannan Street sewer does not conform closely to the grades of the sewers which it intersects. These intersected sewers are generally from 1 to 3 ft. higher than the bottom of the Brannan Street sewer, which thus should become a recipient of the sewage from the entering sewers. As a matter of fact, it is found that the Brannan Street sewer was constructed so carelessly that, where in tunnel, under the south slope of Rincon Hill, there is an offset of 10 or 15 ft., and its bottom grade for several blocks slopes the wrong way. The accumulation of silt in the sewer obliterates the non-conformity of grade with the intersecting sewers and there is therefore a partition of flow at practically every such sewer crossing.

The Brannan Street sewer discharges into the bay under the First Street wharf.

Other sewers on Mission, Howard, and Folsom Streets, have independent outfalls, under the wharves or piers of the waterfront.

The sewage from the northern slope of the Potrero Hills is discharged by small sewers upon the Mission Bay mud flats or, reaching the Potrero Street sewer, it is delivered into the bay in the Potrero near the Union Iron Works.

Yerba Buena Bay Water-shed.—The name, Yerba Buena, as applied to any section of San Francisco is now practically lost. Yerba Buena Bay, the old harbor of Yerba Buena, the precursor of San Francisco, has long ago been filled in and built upon. The extreme westerly end of the bay, that is to say, the point of greatest indentation, reached Montgomery Street. The name has been used in the various sewer reports to designate the low, flat portion of San Francisco which lies north of Market Street, near the bay, and the water-shed includes the slopes of the hills to the north and the west which send their drainage through this flat area. As a sewer district, though of small extent, it is of particular interest and importance, first, because about 120 acres thereof are of the filled-in character, second, because it embraces the most important business section of the city. In this old part of San Francisco are to be found the most striking examples of sewer construction, without comprehensive plan. Almost every street and alley has its 3 by 5-ft. sewer (see Fig. 2), and at every street crossing there is the typical sewer intersection. North and south streets with level surfaces, that is, without gradient, are no exception to the rule. The street in which the standard brick sewer cannot be found is a welcome exception. Until about 1890 there was an outlet for each east and west sewer at the water front. Then, however, a number of these sewers were diverted on East Street from their direct courses and a common outfall at Washington Street was provided for Market Street sewer and the sewers as far north as Jackson Street. This interception of the sewage from the west, which includes the intersection by the lower Market Street sewer of all flow in the sewers from Geary Street to Sacramento Street, makes the Washington Street outlet a point at which a large quantity of sewage is discharged. This outlet is at the low-water line, under the wharf.

The sewers of Vallejo and Pacific Streets have been turned into the

Broadway sewer, which thus becomes their common outlet. It, also, discharges under a wharf at the low-water line. It follows, from the conditions indicated by this description, that much slush and filth has found lodgment in the sewers of this flat area; and it may be added that the gradual subsidence of a part of the area, and the construction of sewers, a block or two at a time, which did not conform to the other sewers in grade, sometimes being several feet too high or too low, have greatly augmented the evil.

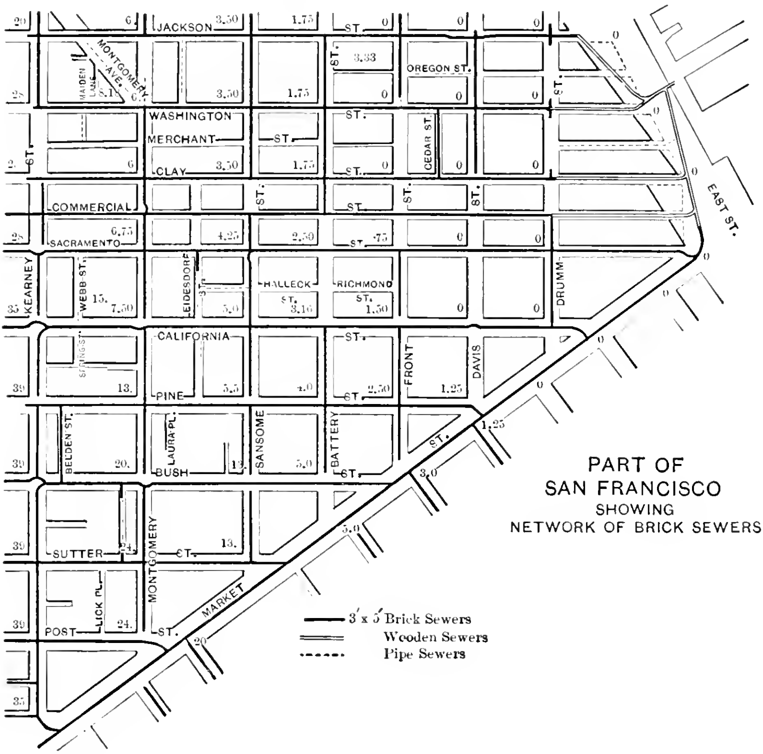


Fig. 2.

The conditions, as they were found when the first examination of this district was made, in 1892, plainly pointed to the use of the existing sewers, in part, at least, as storm-water conduits, and the construction of a new system for sewage only.

North Beach Water-shed.—The North Beach region has a long frontage on the bay. It extends from Telegraph Hill to Black Point.

The main difficulty to be overcome in this region results from the long bay frontage. Sewers now in use discharge into the bay wherever they happen to reach the water line. Sewers of this character are those on Larkin, Hyde, Mason, and Powell Streets.

Washerwoman's Bay Water-shed.—From the Presidio Heights eastward to Russian Hill and Black Point, the northern slope of the northern ridge is toward that part of San Francisco Bay known originally as Washerwoman's Bay. Here, again, the peculiarity of a steep descent from the ridge and the large extent of comparatively flat marginal land is to be noted. Some of the land along the bay remains to be reclaimed.

The built-up area is here fast extending downward on the hillsides toward the bay. Sewers for a long series of years terminated in land-locked basins or at the inshore limit of the tidal marsh. Some of the principal sewers have more recently been extended to the bay shore. The present outfall points of this district are at the bay shore on Pierce Street, on Steiner Street, on Fillmore Street, under a wharf, and on Webster Street.

Lobos Creek.—Lobos Creek had its head originally in Mountain Lake. Though not fed by any well-defined natural lines of surface drainage, it includes within its water-shed all the Richmond District. The southeastern limit of its water-shed is at Lone Mountain. The plateau region northward from the Lone Mountain ridge, westward from First Avenue, and southwestward from the Presidio ridge is drained by this creek. For convenience of classification, all the ocean front drainage, from Fort Point to Point Lobos, may be included in the Lobos Creek water-shed.

The principal sewer of this district is the Richmond main. Here, at variance with the rest of San Francisco, an orderly arrangement of the sewers, which are of comparatively recent construction, may be noted. The outfall of the Richmond main is just west of Baker's Beach, at the foot of Twenty-seventh Avenue.

The Ocean Beach Water-shed.—This region fronts on the ocean, and extends from Point Lobos to Laguna de la Merced. It is without natural lines of drainage, except the small gullies on the western slope of the Twin Peaks group of hills which are soon lost in the deep irregular sand dunes which have drifted in from the ocean.

Laguna de la Merced Water-shed.—Lake Merced lies in the south-

western portion of the city, but a few feet above sea level. The lake has a surface area of about 300 acres. Of its total water-shed of 7 500 acres, somewhat more than one-half lies south of the city in San Mateo County.

The lake is still in use as a source of water for San Francisco, being fed by the waters which percolate through the sands covering a large portion of its water-shed. Some 2 700 acres, of which about 2 300 lie within San Francisco, are owned by the Spring Valley Water Company, around the lake, for its protection against pollution.

The surface run-off from the south is intercepted and turned through a tunnel into the ocean. Likewise, the storm-waters of the Ocean View region are brought within reach of the same tunnel by a long flume. The sewage of Ocean View is carried in a line of cast-iron pipe to an outfall into Merced Creek below the lake.

Studies for a Sewer System and its Design.

The need for a comprehensive system of sewers in San Francisco was pointed out, with increasing frequency and emphasis, by various city officials and committees, until in 1892 the Board of Supervisors, after conference with Frank Soulé, M. Am. Soc. C. E., of the University of California, passed the following resolution, dated March 7th, 1892:

"*Resolved*, That Professor George Davidson of the United States Coast and Geodetic Survey, Col. George H. Mendell of the United States Engineers, and Irving M. Scott, Esq., be and are hereby empowered to select and appoint two engineers, and when the same are so appointed that they, with the gentlemen named, shall constitute a Board of Engineers on and from September 1st, 1892, to devise and provide a system of sewerage for this city and county, as proposed in the report of the Committee on Streets, etc., of this Board, made January 25th, 1892."

At a meeting on August 8th, 1892, the first three members of the Board of Engineers then created:

"*Resolved*, That Marsden Manson* and C. E. Grunsky be appointed as the two civil engineers to complete the Board of Engineers, as contemplated by Resolution No. 6612, adopted by the Board of Supervisors, March 7th, 1892; provided, they agree to devote their entire time to this work after being officially notified by Professor Davidson."

It quickly became apparent to the Board of Engineers thus created, that there were no adequate records of the sewers in use. The sewers

* Member, Am. Soc. C. E., and now City Engineer of San Francisco.

had for the most part been constructed piecemeal, a block or a street crossing at a time, and diagrams to illustrate the work had frequently been considered superfluous. In many cases no record had been made of the completed structures. It was necessary, therefore, to make an examination of the sewers by entering manholes and by exploring the larger sewers. In this way dimensions and conditions as to serviceability, if retained in use as a part of a comprehensive system, were ascertained. Several exploring parties were organized for this purpose, and the results of their work were at once mapped on a suitable scale.

The preparation of a complete topographic map was also undertaken and this was subsequently found to be of great value in facilitating the studies.

Attention was at once given by the Board to the selection of suitable points of outfall for the city's sewage. It was assumed at the outset that only one method of disposal—discharge into the bay or ocean—would come under serious consideration. Therefore, plans were made to observe the direction and velocity of the bay currents.

The work of the Board of Engineers had not been far advanced before its independent course gave offense in certain quarters and led to its dismissal (January, 1893). Its books and the data collected in the four months of activity were by the Board of Supervisors turned over to Mr. Marsden Manson who, however, declined an offer to continue the work. A few days later, after repeated conferences with members of the Board of Supervisors, Mr. Manson and the writer were appointed as engineers in charge to devise and provide the sewer system.

The work outlined by the Board of Engineers was continued, and has become the basis for all sewer work in San Francisco since that time. On July 1st, 1893, the existing sewers, 200 miles in length, had been examined and their general condition noted. All sewers through which it had been possible for a man to pass had been examined as to general condition. In some instances, where data could not be obtained otherwise, the sewers were uncovered. The ascertained facts were tabulated for convenient reference, and were platted on sheets of convenient size on a scale of 100 ft. to the inch. Sixteen of these sheets were at that time complete, ten were in progress, and forty more would be required to cover the built-up areas of the city. At this time, the general map of the city, on a scale of 600 ft. to the inch, had been nearly completed.

Float observations at all stages of the tide were made in the bay from a number of points, principally Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. The purpose of these observations was to make sure that the discharge of sewage at the outfall points, as finally selected, would, by reason of ample dilution, become inoffensive and harmless before any of it drifted inshore.

The work of the Engineers in Charge, however, was not carried forward to the completion of a design of a system of sewers. They shared the same fate as the Board of Engineers—summary dismissal in the fall of 1893—and nothing further was done until 1899 when the Board of Supervisors determined to continue the work under the same engineers but with the addition of the City and County Surveyor, the late Charles S. Tilton; this addition being made as a matter of policy. The resolution authorizing the continuance of the work was passed in May, but, owing to some uncertainty regarding its legality, it had to be re-enacted in modified form. This took time, and it was not until late in September that it seemed certain that the funds set apart for the work would become available. In the meanwhile the work was under way and was rapidly pushed to completion before the close of the year.

Before submission to the Board of Supervisors the report of 1899 on the sewer system was passed upon by Rudolph Hering, M. Am. Soc. C. E., and received his endorsement.

Some Features of the Problem.

The question of determining whether sewage and rain-water should be carried in the same or separate conduits was necessarily one to be determined for each of the many districts into which the city is divided by its topographic features. The method and place of disposal and the sewers already in service were elements requiring to be weighed in this determination. It was also necessary to make a careful study of rainfall records, in order to ascertain the volumes of storm-water flow that would have to be passed through the sewers to the bay or ocean.

Before taking up the discussion of these matters some additional notes relating to the old sewers, to show the general lack of system in their arrangement, may not be out of place.

The statement, that the plan for providing sewers seems to have been to construct egg-shaped brick sewers, 5 ft. high and 3 ft. wide, in

all streets and alleys, where property was valuable and could afford to pay for large sewers, as applying to most of the sewer work that had been done in San Francisco, is substantially true. The size of sewer required was frequently determined by the Superintendent of Streets, who was never a civil engineer, and the prescribed depth of the sewer, as defined by ordinance, was 10 ft. below the official grade of the street. Therefore, sewers are found, as already explained, where they are all but useless, and types of construction have resulted that are certainly unique. These facts were set forth at some length in the Progress Report of 1893 of the 'Engineers in Charge.'

For years the ordinary improvement of street crossings included sewers across the streets in both directions and storm-water inlets at the curbs on the four corners. The sewers in such intersections were built of brick—four wings from a central manhole. The invert, as required by ordinance, was placed 10 ft. below street grade, generally level, or, due to the intelligence of most of the sewer contractors, a few inches low at the down-hill side of the street intersection. The sewers in the intersection might connect with other brick sewers of like size, or with larger sewers or with small pipe sewers, according to what was prescribed at some other time for the streets leading from the intersection. After a time, there was a modification of this arrangement of sewers at intersections of streets. Where it was known that the property owners of outlying districts could not afford the large brick sewers, pipe sewers were prescribed for the crossings. The "Bobtail" crossing was introduced. This consists of a section of 5 by 3-ft. sewer 16 ft. long placed along the axis of one street with pipe sewers from its ends and with pipe sewers along the axis of the cross street. A final modification for crossings of small sewers consisted in omitting altogether the brick sewer and putting in two practically horizontal pipe sewers along the center lines of both intersecting streets.

There is no excuse for the many useless storm-water inlets that have been constructed on gutter summits. These occur in all parts of the city which had been improved to any extent before 1890. In 1893, in a central portion of the residence district of the city, a count was made of the storm-water inlets located at corners from which there was a down gradient in both directions, into which, therefore, no water can flow. The district covered by the count extended from Taylor Street on the east to Broderick Street on the west, and from Washington

Street on the north to Hayes Street on the south. In it there are 1 080 storm-water inlets, of which 149 are absolutely useless, and many more receive water from so small an area of street surface that they are to be classed as unnecessary. Each of these useless and unnecessary storm-water inlets, or catch-basins, as usually called, cost the property owners about $90.

In San Francisco, as in most of our cities, the improvement of streets and the construction of local sewers is paid for by the property which fronts on the improvement. The work may be done in districts, as was done in the Richmond District, when the main lines of drainage were put in, and in the case of the Army Street sewer; but, ordinarily, only short stretches of sewer were covered by the official order of construction or in advance of such order, under which there would be a public letting of the work to the lowest bidder, the property owners would be prevailed upon to enter into a private contract to do the work. The favored contractor was at times responsible for the size of the sewer, suggesting it and securing official approval; at other times the Superintendent of Streets and Sewers determined the character and size, at any rate, this matter was subject to his approval. Only in the minority of cases was it possible to refer to a general plan covering some section of the city for sewer sizes.

The city, preceding 1900, had a City and County Surveyor, whose principal duties related to lot surveying. This was an elective office. As the law also required that a City Engineer be appointed by the Board of Supervisors, it was customary for the Board of Supervisors to appoint the incumbent of the elective office to the office of City Engineer. For services rendered in the capacity of Surveyor or Engineer fees were collected, and when special work of any kind was required by resolution or order of the Board of Supervisors, the compensation to be made was usually named therein.

Supervision of construction was in the hands of the Superintendent of Streets. The City Engineer staked out line and grade and issued a certificate, showing satisfactory construction or showing by diagram departures from prescribed alignment or grade. He was not held responsible for the acceptance of work, however great the deviation from a required position might be.

This unsatisfactory method of prescribing and carrying out street and sewer work continued until the present city charter went into

effect, in January, 1900. Since that time a Board of Public Works has had charge of street and sewer construction, and under a well-organized Bureau of Engineering, with a salaried City Engineer at its head, all matters relating to sewer design and construction receive competent and proper attention.

A few diagrams were prepared in 1893 by the Engineers in Charge, to illustrate the lack of system, and peculiarities of the treatment of sewerage problems in San Francisco. Of these only one, Fig. 2, showing the almost universal use of 3 by 5 ft. brick sewers, is exhibited herewith.

Rainfall in San Francisco.

When the studies relating to the design of a comprehensive sewer system for San Francisco were commenced in 1892, an inquiry was at once made of the Weather Bureau for data relating to rates of precipitation. All that could then be learned was that there were no rains of an inch or more per hour, and that therefore the Weather Bureau had not undertaken to record the rate of rain. But it was agreed that the matter should at once be taken up, as it was quite as important for San Francisco to have positive evidence that the maximum rain in an hour was about 0.5 in. as for the cities of the Atlantic Slope to know that 2 in. or more per hour may be expected occasionally.

Therefore, in 1899, when the matter was taken up again, enough information was at command to enable satisfactory conclusions relating to maximum rain intensities to be reached. Mr. Thomas Tennent, a retired maker of nautical instruments, who had for nearly 50 years kept a rainfall record, was of assistance in the rainfall study. His records were placed at the writer's disposal* and were freely used. From them and from those of the Signal Service and United States Weather Bureau (commencing March 9th, 1871), it was found that there had been rain in excess of 2 in. per day as shown in Table 1.

In order that the infrequency of the days on which there is rain in excess of 2 in., and their occurrence, unless as a rare exception, only in the months November to April, may be understood, some further notes on rainfall and climate may not be out of place. Very little rain is expected from May 1st to November 1st. There are no violent

* At the writer's suggestion, these records were turned over by Mr. Tennent to the U. S. Weather Bureau. They were lost in the fire of 1906.

thunderstorms, as in the States of the Atlantic Slope, the Middle West and the Central Northwest. Even in the winter, rain storms accompanied by thunder and lightning are of rare occurrence.

TABLE 1.—RAINFALLS, AT SAN FRANCISCO, IN EXCESS OF 2 INCHES PER DAY.

Date.	Inches.	Date.	Inches.	Date.	Inches.	Date.	Inches.
Jan. 29, 1850....	2.20	Jan 9, 1862....	3.50	Feb. 20, 1867....	2.12	Dec. 23, 1884...	2.01
Dec. 17, 1852....	3.00	Jan. 10, 1862....	2.46	Feb. 21, 1867....	2.22	Nov. 23, 1885...	2.58
Dec. 25, 1852....	2.54	Jan 16, 1862....	2.46	Dec. 18, 1871....	2.83	Dec. 21, 1885...	2.78*
Jan. 7, 1853....	2.06	Jan. 17, 1862....	2.64	Dec 19, 1871....	3.12	Jan 23, 1886...	2.35
Mar. 27, 1853....	2.85	Feb. 21. 1862....	2.09	Dec. 23, 1871....	2.48	Feb. 4. 1887...	2.22
Apr. 16, 1853....	3.45	Nov. 26, 1864....	3.98	Jan. 8. 1872....	2.36	Feb. 5. 1887...	2.92
Mar. 13. 1854....	2.25	Dec. 12, 1864....	2.56	Nov. 29, 1872....	2.06	Mar. 13, 1889...	2.54
Oct. 21. 1858....	2.06	Jan. 17. 1866....	2.17	Nov. 23, 1874....	3.98	Feb. 15. 1891...	3.38
Mar. 26, 1861....	2.53	Jan. 20, 1866....	2.22	Nov. 17. 1875....	2.30	Dec. 29, 1891...	2.21
Dec. 26, 1861....	2.02	Dec. 19, 1866....	4.28	Mar. 5, 1879...	2.73	Nov. 23, 1896...	2.00
Jan. 5, 1862....	2.67	Dec. 20, 1866....	3.62	Jan. 29, 1881....	4.67	Mar. 22, 1899...	2.15

* In 11 hours.

The cyclonic disturbances that pass over the country from west to east—usually from two to four per month—are generally of great extent. When they are accompanied by rain, there is ordinarily a steady downpour at a moderate rate until the area of low barometer has passed on to the eastward, when the change of wind direction from southeast to southwest and the fall in temperature may be accompanied by heavy clearing-up showers. These showers, which are not frequent, are generally local. They may cover only a small portion of the area of San Francisco, and generally have sharply defined limits. Even when the entire city is shower swept, the heavy fall of rain is not simultaneous in all parts of the city. Neither are the maximum rates of rain long sustained. The fact that the maximum rainfall in one hour is well below 1 in. is clearly established by all the records of rain that have been kept in San Francisco. On this point, under date of June 29th, 1899, Mr. Thomas Tennent says:

"In reply to your inquiry concerning the rainfall in San Francisco, and what has been the greatest quantity which has fallen in a given time, say one hour, I will state that I have personally kept a record of the rainfall in this city from 1849 to 1896, measuring the quantity in each 24 hours at 8 A. M. On looking over my records [preceding the establishment of the U. S. Signal Service in March, 1871] I find that the heaviest rainfalls we have had in 24 hours occurred, viz.: 1864,

November 26th, 3.98 in., with the remark, 'Heavy rains in the morning; rain and gale at midday; steady rain P. M. and evening.' 1866, December 19th, 4.28 in., 'heavy rain and gale in the morning; hail with thunder at midday, rain during the balance of the day and evening.' 1866, December 20th, 3.62 in., 'high wind and cloudy in the morning; then a steady rain all afternoon and evening.' I can state without hesitation, that we have never had a fall of rain of 1 in. in any one hour."

Based on the records available in 1899 it was found that:

The average number of rainy days per year is 66. There are to be expected no days on which it rains 5 in., and the probabilities are:

1 day in 50 years with more than	4.50 in.
1 " " 25 " " " "	4.00 "
1 " " 10 " " " "	3.50 "
1 " " 6 " " " "	3.00 "
2 days " 5 " " " "	2.50 "
1 day per year " " "	2.00 "
2 days " " " " " "	1.50 "
6 " " " " " " "	1.00 "
9 " " " " " " "	0.75 "
16 " " " " " " "	0.50 "
26 " " " " " " "	0.25 "
40 " " " " " less than	0.25 "

The frequency of rain and the number of days on which rain fell in excess of the several amounts above named is shown by the diagram, Fig. 3, as determined by actual count for a period of 50 years, from 1849 to 1899. The close agreement between the fact and the probability, indicated by the curve, is noteworthy.

From this study it was concluded that rain storms with excessive fall of rain are rare; that nearly two-thirds of the rainy days are days with less than ¼ in. of rain; and that any day with rain in excess of 5 in. is highly improbable.

It should be stated, too, that on the days that show light rain the precipitation is usually well distributed, the rain is light and continuous during the greater part of the 24 hours, more frequently than in the form of showers of short duration.

The data secured by the U. S. Weather Bureau subsequent to the inquiry of the Board of Engineers in 1893, relating to rain intensity,

were carefully studied. As already stated, there had been a sufficient number of heavy rains to enable satisfactory conclusions to be reached.

On	January 20th, 1894,	it	rained	0.15	in.	in	5	min.
"	October 12th, 1899,	"	"	0.09	"	"	"	"
"	November 23d, 1896,	"	"	0.08	"	"	"	"
"	January 20th, 1894,	"	"	0.17	"	"	10	"
"	November 23d, 1896,	"	"	0.14	"	"	"	"
"	November 23d, 1896,	"	"	0.55	"	"	1	hour
"	January 20th, 1894,	"	"	0.36	"	"	"	"

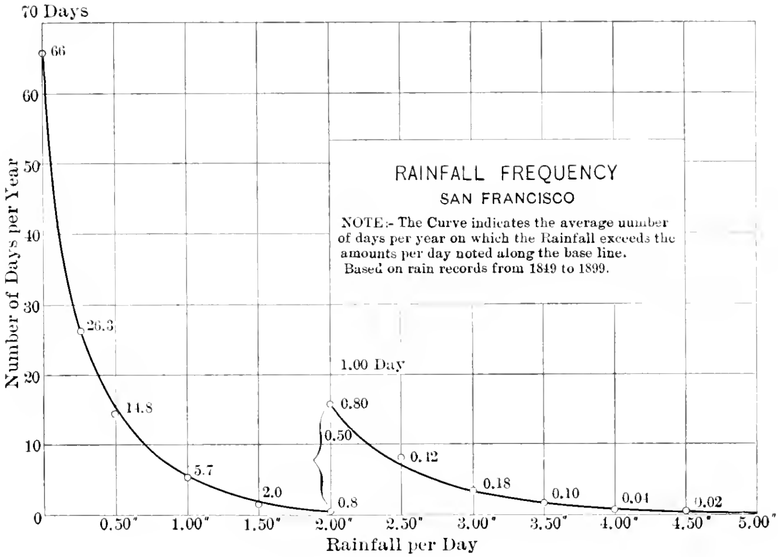


FIG. 3.

Using these and other data from the records, a curve of rainfall intensity was prepared, or rather a limiting curve to show the maximum amount of rain to be expected in any time unit up to 24 hours. The lower portion of this curve was platted on an enlarged scale. This curve is shown by Figs. 4 and 5.

Based on rainfall measurements made subsequent to 1899, a new curve has come into use, which shows somewhat greater rain intensities for short time-periods. The curve now in use is shown in dotted lines.

The most important fact relating to rain intensity disclosed by

the U. S. Weather Bureau records since 1899* is that there was one rain storm in the spring of 1904 which exceeded the intensities indicated by the curve now in use, by about 10% for all periods of time up to one hour. Only two other rain storms were of greater intensity than those indicated by the curve, but the excess in one case was only for a 5-min. period, in the other for a 10-min. period. It is believed that the heavy rain in the spring of 1904 did not cover the entire city.

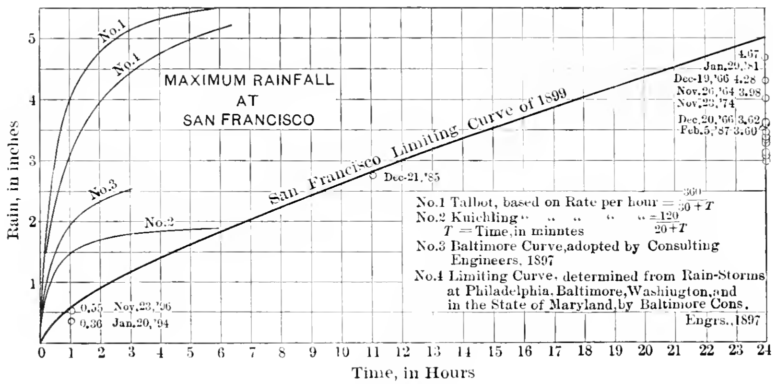


FIG. 4.

The interpretation of the diagram prepared in 1899 established the following for San Francisco:

The maximum amount of rain to be expected:

In	any	5-min.	period	is	0.15	in.
"	"	10 "	"	"	0.22	"
"	"	20 "	"	"	0.32	"
"	"	30 "	"	"	0.40	"
"	"	40 "	"	"	0.47	"
"	"	50 "	"	"	0.54	"
"	"	60 "	"	"	0.60	"
"	"	2-hour	"	"	0.90	"
"	"	3 "	"	"	1.15	"
"	"	6 "	"	"	1.83	"
"	"	12 "	"	"	3.00	"
"	"	24 "	"	"	5.00	"

* From facts furnished by H. De H. Connick, Assoc. M. Am. Soc. C. E., Assistant City Engineer.

It was confidently assumed that these quantities of rain would rarely be exceeded, and, if exceeded, that the area covered by the excessive rain would probably be small; that therefore these quantities might safely be accepted in estimating maximum storm-water run-off.

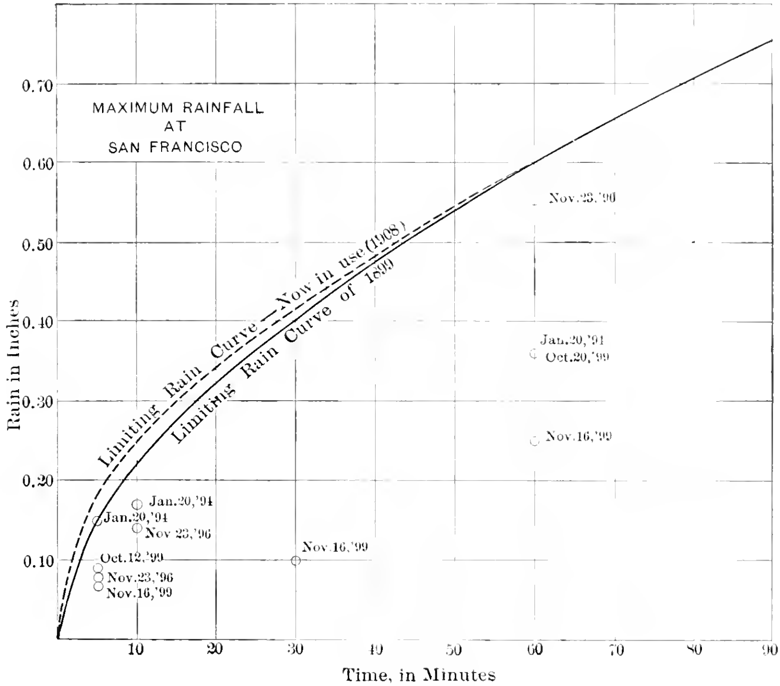


FIG. 5.

Bearing in mind the fact that a rain expressed in inches per hour is practically equivalent to the same number of cubic feet per second per acre, it follows from the above that it may rain in San Francisco as follows:

Duration, in minutes.	Rate, in inches per hour.	Equivalent, in cubic feet per second per acre.
5	1.80	1.80
10	1.32	1.32
20	0.96	0.96
30	0.80	0.80
40	0.70	0.70
50	0.65	0.65
60	0.60	0.60

This is represented graphically in Fig. 6. The ratios above noted were used in estimating the required capacity of storm-water sewers. They have been modified somewhat since 1899, as stated, for the shorter time periods, so that at present, when a sewer for a small district is designed, somewhat larger values for storm-water flow are obtained.

For comparison, the limiting rain curve now in use is shown as already stated in Fig. 5, and the corresponding maximum rainfall rates in Fig. 6.

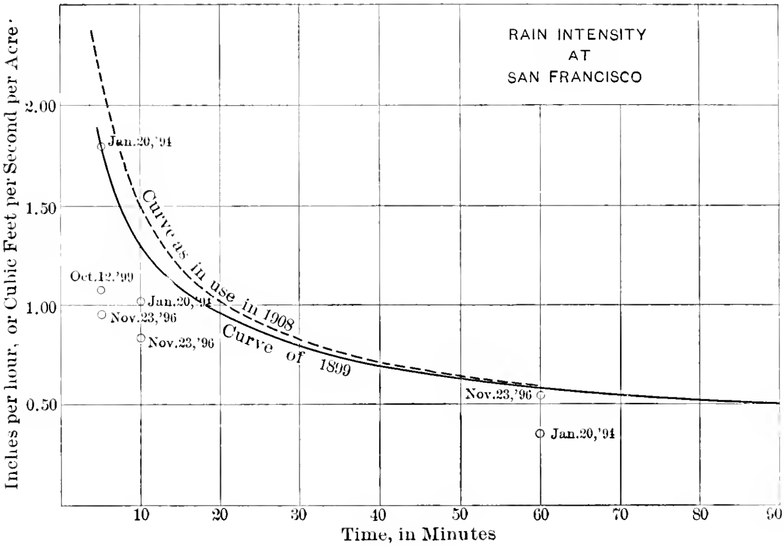


FIG. 6.

Table 2 is the rainfall rate table now in use.

TABLE 2.—RAINFALL INTENSITY AT SAN FRANCISCO, AS IN USE IN 1908.

Time, in minutes.	Intensity, in inches per hour.	Time, in minutes.	Intensity, in inches per hour.
5	2.16	20	1.02
6	1.97	30	0.83
7	1.81	40	0.72
8	1.69	50	0.65
9	1.59	60	0.60
10	1.50	70	0.56
15	1.20	..	....

In order that the great disparity between the maximum rainfall rates to be expected on the Pacific Coast and those of the Atlantic

Slope may become apparent, Fig. 4 shows several other limiting rain curves, in part taken from the report of the Board of Consulting Engineers for the City of Baltimore.* The limiting rain curve adopted by this board for Baltimore, and also a limiting curve determined by rainfall in Washington, D. C., in Philadelphia, Baltimore, and in the State of Maryland, are reproduced in the diagram, and also the limiting rain curves suggested by A. N. Talbot, M. Am. Soc. C. E., and by Emil Kuichling, M. Am. Soc. C. E.

The rainfall intensities indicated by the San Francisco curve of 1899 can be expressed by a formula of the type

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{0.4}} \qquad \text{(21)}$$

Here I is the greatest possible mean rate of rainfall (the maximum intensity), expressed in inches per hour, when t represents the duration of the downpour, in minutes. The numerator, b, is a constant for any locality, but will vary within wide limits for different localities.

For San Francisco, $b = 3.68$.

For points on the Atlantic Slope, b will generally have a value lying between 8 and 12.

In Table 3 rain intensities are noted for several values of b, and, for comparison, the rain intensities as a deduced from the limiting rain curve of 1899 are also noted.

The Estimate of Storm-Water Flow.
Method of 1899.

Having determined the maximum rainfall rates, some method had to be adopted of estimating from these rates, the maximum rate of flow to be expected at any point of the storm-water conduits. Such method must of course be applicable to areas of all manner of shapes and sizes, of varied surface topography, and of varied character so far as determined by the improvements, present and prospective.

It is obvious that the maximum rate of flow at any point of a storm-water conduit will be less than the maximum rate of rainfall upon the tributary area. Some of the rain will be taken up by porous surfaces, some of it will evaporate, and at the time that the flow is a maximum much of the rain-water will still be scattered over the

* Report of Sewerage Commission, City of Baltimore, 1897.

surface or will be in the conduits the filling of which retards the passage of and elongates the flood wave.

TABLE 3.—Maximum Intensity of Rainfall, As Determined in 1899 for San Francisco, and Based on the Formula:

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{0.4}}$$

Duration of rain, in minutes	San Francisco.			Atlantic Slope.	
	From curve of 1899. Inches per hour.	By formula: $b = 3.68$ Inches per hour.	By formula: $b = 4$ Inches per hour.	By formula: $b = 8$ Inches per hour.	By formula: b = 12 Inches per hour.
5	1 80	1.80	1.95	3.90	5.85
10	1.30	1.32	1.43	2.86	4.28
15	1.10	1.10	1.19	2.38	3.57
20	0.95	0.97	1.05	2.10	3.15
30	0.80	0.81	0.87	1.75	2.62
40	0.70	0.72	0.77	1.55	2.33
50	0.65	0.65	0.70	1.40	2.11
60	0.60	0.60	0.65	1.30	1.95
90	0.50	0.51	0.55	1.10	1.65
120	0.45	0. 15	0.49	0.98	1.47
180	0.38	0.387	0.42	0.84	1 26
240	0.34	0.350	0.38	0.76	1.14
1 440	0.20	0.182	....	....	....

Experience in other cities indicates that the combined effect of these causes, which contribute to make the rate of flow in the conduit less than the rain rate, can with some degree of approximation be expressed as a percentage based upon the proportional part of the drainage area which is impervious. This fact has been pointed out by Mr. Emil Kuichling,* and also the further fact that density of population may be used as a guide in determining the correction factor to be used.

It was entirely out of the question to make experiments in San Francisco to be used in the approximation of reduction factors. The assumptions made, therefore, were based upon the experience in other cities. In San Francisco, as has been explained, the demarkation of the area tributary to any storm-water main is not possible without first modifying the sewer crossings. But even if it had been possible to prepare one or more areas for observation, it would have been folly to do

*Transactions, Am. Soc. C. E., Vol. XX, p. 1.

this in the hope that some storm might occur within the short time available for preparing the report, particularly as results were wanted before the next wet season. It is to be remembered, too, that such results would have been of slight value, because they would have represented run-off rates for conditions as they prevailed in 1899 and not for those that will prevail in the future.

The study of population distribution was based largely upon the records in the Registrar's office. From the size of voting districts and the number of votes in each, with due allowance for the class of people inhabiting each, an estimate of population density was made. Based on this study, it seemed proper to regard all parts of the city having a population of 100 or more per acre as impervious. A population density was then assumed for each sewer district such that the aggregate in all districts would be 1 000 000.

Table 4 shows the reduction factors finally used in estimating the required storm-water capacity from the rate of rainfall.

TABLE 4.

Population per acre.	Reduction factor.	Population per acre.	Reduction factor.
20	0.30	70	0.65
30	0.40	80	0.70
40	0.50	90	0.73
50	0.55	100	0.75
60	0.60	...	....

The procedure in determining the required storm-water capacity for any point of a conduit was substantially as follows: The time was approximated which would elapse before the water falling as rain in the most remote part of the district would reach the point at which the capacity was to be determined. Distance and fall between governing points along drainage lines enabled a quick and sufficiently close approximation of this time to be made. From 3 to 5 min. were then added as the time allowance for the rain-water to collect in gutters and flow to the storm-water inlets. Although within this elapsed time, there may have been shorter periods with higher rates of rainfall, it is to be assumed that these were practically coincident throughout the district, and that much of the water falling at the maximum rate upon nearby portions of the area had already passed before the water from more remote sections arrived. Consequently, the average rate of

rainfall during the entire critical period was alone taken into account. This was the maximum rate of rain for the estimated time, as shown by the curve in Fig. 6.

The reduction factor, determined by population density, or otherwise, was then applied. In this way the maximum run-off rate per acre was estimated from point to point for each main sewer. Where two main lines of sewers came together, independent estimates were made for each, and the results were combined, if the two districts were not too dissimilar in shape and extent. Otherwise, the time determination, as made for the larger district alone, was used in calculating the rate of run-off. This was done because the maximum flow in the drainway of the smaller district would occur earlier than in the larger district and the allowance for required capacity might become unreasonably large.

It is to be understood, that this method of calculating storm-water flow was applied with care. Cases are readily conceivable where the shape of a contributing area is such that it would be improper to apply to the entire area a rain rate determined from the time required by water to flow from the most remote point thereof. The area under consideration may have one or more finger-like projections, long and narrow, which, if taken into account, would make the time long and the rain rate small. In such cases it becomes necessary to examine the lower compact portion of the area by itself. The rain rate is determined in the usual way, and if its application to the partial area, particularly if, owing to surface character, a larger proportion of run-off is to be assumed, gives larger results than the treatment of the district as a whole, then this value determines the required conduit capacity for storm-waters. Sometimes a succession of trials is requisite to establish the maximum possible value.

This method, in its entirety, however, is unsatisfactory, mainly for the reason that it leaves too much latitude to the judgment of the engineer. It has not found general acceptance.

Similar and yet more serious objection may be urged against the use of the various formulas for storm-water flow that have from time to time been suggested by hydraulicians for application to urban areas. None of these, moreover, could have been applied to San Francisco conditions without special study relating to the determination of constants to fit them to local conditions. Even then there would have been grave doubt as to whether formulas predicated on rainfalls exceeding

those at San Francisco from two to six-fold for time periods of one hour and less should be considered applicable. It is only necessary to glance at the rain curves in Fig. 4 to appreciate the force of this statement.

In all the formulas that have come to notice the rate of rain with some suitable exponent, generally unity, appears as a factor. The rain rate to be used in the formulas must be determined by the engineer, whose judgment is guided by the available rainfall data and his past experience. Thereupon a correction factor, based upon the character of the surface of the district, must be adopted. Even when rain data are ample and the constants have been determined with care the result may be rendered more or less doubtful if the district under consideration has an unusual shape, or has exceptional hypsographic features.

These considerations led to the conclusion, in 1899, that it was not practicable to adapt any of such well-known formulas as those of Hawksley, Adams, McMath or Bürkli-Ziegler, to the conditions prevailing in San Francisco.

The method already described, appearing in 1899 to be the most rational then known, was therefore adopted.

But the need was felt, and has not heretofore been met, of some more satisfactory method of translating rain rates into storm-water flow. The result of a further study of this subject can now be presented with the hope that it may prove of some value to the Engineering Profession.

New Method of Estimating Required Capacity of Storm-Water Conduits.

Surface Run-off.—The rain which falls upon any area during the time in which the run-off rate is increasing is in part accounted for by infiltration into porous soils and by evaporation; in part it flows past the outfall point of this area, and in part it has swelled the water contents of the storm-water conduits within the area, or is still in transit across the surface to the storm-water inlets.

It has been stated that the method of estimating sewer capacities, as used in the San Francisco problems, takes into account, after a fashion, the retarding influence of temporary water storage on the ground and in the conduits. The new method makes this storage,

which is capable of approximation, an essential element in the calculation of storm-water flow.

This method is applicable wherever it is possible to determine the volumetric increase of the water actually in the conduits during the progress of a rain of the extreme type. The first step, again, is the determination of the maximum rain rates for increasing time intervals. Thereupon, with these rain rates as a guide, the rates can be determined at which the water will run from the surface of the ground. The final step is the modification of these surface run-off rates by the effect of water accumulation or storage on the surface and within all conduits which are located above the point at which storm-water capacity is to be determined.

It is to be remembered that the percentage correction, heretofore noted as applicable to rain rates when storm-water flow is to be estimated by the method used in 1899, covers, not alone the reduction of flow due to water absorption and evaporation, but also the retarding effect due to temporary water storage. These correction factors, therefore, cannot be used when it is desired to estimate for a relatively short time during the progress of a heavy rainfall how much of the water reaches and enters the storm-water inlets.

The greatest intensity of rain is rarely at the beginning of a storm. For the purpose of estimating conduit capacities it must be assumed to occur at a time when the ground is already wet and no longer capable of absorbing water as rapidly as at the beginning of the rain. The rate of water absorption by porous ground plus evaporation though possibly several times greater at the commencement of a downpour than after the same has continued for some time, by no means keeps pace with the rain rate. It follows that it would not be correct to apply a reduction factor, ascertained from aggregate quantities of rain and of run-off produced by the storm, to the maximum rain rate in order to estimate therefrom the maximum surface run-off rate.

The amount of water which evaporates during the short time covered by the critical periods involved in urban run-off problems, will be relatively small and, for all practical purposes, may be disregarded.

The run-off, consequently, from an impervious area, the highest supposable type of urban area, during any critical period, measured on the ground where the rain falls, may be regarded as equal to the

maximum rain intensity for that period; and the rate of inflow into storm-water inlets will be this run-off rate, modified only by the effect of changes in the volume of water temporarily stored on the surface of the area.

It will be shown, hereinafter, that the amount of water actually lying upon or flowing over the exposed surfaces of the area under consideration cannot be materially different, at the end of a critical time period, from the amount thus covering the area at the beginning of the period, except when the critical time period is of short duration. It may be neglected whenever the critical time period exceeds 15 min.

The amount of water in temporary storage on the ground's surface, that is, the water momentarily in transit, on roofs, on lawns, in gutters and otherwise, to the storm-water inlets, can be estimated if the time is known that it will take water during heavy rains to flow to the inlets from the outer portions of the small areas served by the individual storm-water inlets. This time is usually from 5 to 10 min. Make the unfavorable assumption that it is 5 min.

The area tributary to each inlet may now be regarded as subdivided into 5 concentric zones, from the innermost of which water will reach the inlet in the average time of $\frac{1}{2}$ min., from the next in $1\frac{1}{2}$ min., and so on to the last one in $4\frac{1}{2}$ min.

At the end of any 5 min., supposing the rain to fall at a uniform rate during this time, there will still be on the ground, flowing toward the inlet $\frac{4.5}{5}$ in. of the rain which fell on the outer zone, $\frac{3.5}{5}$ in. of the rain which fell on the next smaller zone, and so on down to the innermost zone, on which there will still be one-half of the rain which fell during the last minute. (No correction has been made for the fact that the outer half, by time, of each zone is somewhat larger than the inner half.)

If, now, the area which is served by an inlet be considered as circular, with the inlet in the center of the area, then the areas of the successive zones and the entire area will be to each other as 4, 12, 20, 28, 36, and 100.

Calling the total amount of rain which fell upon the area in 5 min. $100x$, then the amount of water remaining on the ground at the end of this period will be

$$\left(\frac{4}{10} + \frac{36}{10} + \frac{100}{10} + \frac{196}{10} + \frac{324}{10}\right) x = 66x$$

The water on the ground at the end of the 5-min. period, therefore, is $\frac{66\,x}{100\,x}$, or two-thirds of the total amount of rain which fell in that period.

Generalizing, it may be said that what is true of an area with circular outline is true of other compact shapes, and that, therefore, two-thirds of the water which falls during any time period equal to the time required by water to reach the inlets from the outer portions of areas drained by each, will, at the end of that period, be still on the surface in transit to the inlet.

For any limiting rain curve, therefore, the amount of water on the ground in a district with impervious surface can be estimated for any instant of time, if the time is known that will be consumed by the water in flowing from the limits of an inlet area to the inlet.

For the inlet or entrance time equal to 5 min., and the rain rates indicated by the limiting rain curve of 1899 for San Francisco, the amount of water on the ground at the end of successive intervals of 5 min. is shown in Column 5 of Table 5.

TABLE 5.—WATER ON THE GROUND DURING RAINFALLS OF EXTREME INTENSITY AT SAN FRANCISCO.

Based on Rain Intensities as determined in 1899, using the formula:

$$I = \frac{3.68}{\frac{2\,t}{t + 60} + t^{0.4}}$$

Successive periods: Duration, in minutes.	Greatest possible total rain to the end of the period, in inches.	Rain during each successive period, in inches.	Intensity of rainfall, in inches per hour; or, approximately, second-feet per acre.	Water on the ground at the end of each period, in cubic feet per acre.	Greatest possible total rain to the end of each period, in cubic feet per acre.	$G_T - G_{T+5}$ cubic feet per acre.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0 to 5	0.150	0.150	1.795	360	540	....
5 to 10	0.219	0.069	0.828	166	790	195
10 to 15	0.275	0.056	0.672	134	1 000	32
15 to 20	0.321	0.046	0.552	112	1 170	22
20 to 25	0.364	0.043	0.516	103	1 320	9
25 to 30	0.404	0.040	0.480	95	1 470	8
30 to 35	0.441	0.037	0.444	89	1 610	6
35 to 40	0.476	0.035	0.420	84	1 730	5
40 to 45	0.509	0.033	0.396	79	1 830	5
45 to 50	0.541	0.032	0.374	75	1 970	4
50 to 55	0.571	0.030	0.360	72	2 080	3
55 to 60	0.600	0.029	0.348	70	2 180	2
60 to 120	0.906	0.306	0.306	61	3 290	....
120 to 180	1.158	0.252	0.252	50	4 200	....
80 to 240	1.396	0.238	0.238	48	5 090	....

TABLE 6.—REDUCTION FACTORS WITH WHICH TO MULTIPLY MAXIMUM RAIN RATES TO FIND MAXIMUM SURFACE RUN-OFF RATES.

The value of the constant, a, in $r = a\,I$

$$\text{When } I = \frac{b}{\dfrac{2\,t}{t + 60} + t^{0.4}}$$

Duration of the critical period, in minutes.	San Francisco. $b = 3.68$ $R = 0.60$ $c = 0.0775$					$b = 4$ $R = 0.65$ $c = 0.084$					$b = 8$ $R = 1.30$ $c = 0.169$					$b = 12$ $R = 1.95$ $c = 0.253$				
	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
5	1.00	0.91	0.81	0.71	0.61	1.00	0.91	0.82	0.73	0.64	1.00	0.96	0.91	0.87	0.82	1.00	0.97	0.94	0.91	0.88
10	1.00	0.88	0.76	0.64	0.51	1.00	0.89	0.77	0.66	0.54	1.00	0.94	0.88	0.83	0.77	1.00	0.97	0.93	0.89	0.85
15	1.00	0.86	0.71	0.57	0.42	1.00	0.87	0.74	0.60	0.47	1.00	0.93	0.87	0.81	0.74	1.00	0.96	0.92	0.87	0.83
20	1.00	0.84	0.68	0.53	0.37	1.00	0.86	0.72	0.56	0.43	1.00	0.93	0.86	0.79	0.72	1.00	0.96	0.91	0.86	0.81
30	1.00	0.83	0.66	0.49	0.31	1.00	0.85	0.69	0.53	0.37	1.00	0.92	0.84	0.76	0.69	1.00	0.95	0.90	0.85	0.79
40	1.00	0.82	0.64	0.46	0.27	1.00	0.84	0.67	0.51	0.34	1.00	0.92	0.83	0.75	0.67	1.00	0.95	0.89	0.84	0.78
50	1.00	0.81	0.63	0.44	0.25	1.00	0.83	0.66	0.49	0.31	1.00	0.91	0.82	0.74	0.65	1.00	0.94	0.88	0.83	0.77
60	1.00	0.81	0.62	0.43	0.23	1.00	0.82	0.65	0.47	0.29	1.00	0.91	0.82	0.73	0.64	1.00	0.94	0.88	0.82	0.76
90	1.00	0.80	0.59	0.38	0.18	1.00	0.82	0.63	0.44	0.25	1.00	0.91	0.82	0.72	0.63	1.00	0.94	0.88	0.81	0.75
120	1.00	0.79	0.58	0.37	0.16	1.00	0.81	0.62	0.42	0.23	1.00	0.91	0.81	0.71	0.62	1.00	0.94	0.87	0.80	0.74

The absorption of water by the soil will vary directly with the proportional part of the surface that is pervious, and will also vary with the character of the soil and subsoil. It will be largest in districts having deep soils of the sand or gravel type that take an indefinite quantity of water freely, and will be least in districts with heavy clay soil or with a thin soil cover on impervious rock formations.

The rate at which the water thus deflected from a course across the surface of the ground will sink into the soil should be assumed to be a gradually decreasing rate. Soil will take water somewhat more readily at the beginning of a downpour than after a heavy rain has continued for some time.

This rate is determinable, and upon it will depend the relative amount of the rain which will from time to time, during the progress of a heavy rain, reach the storm-water inlets.

Data have not been accessible for an entirely satisfactory determination of reduction factors to be applied to rain rates to reduce the rain rates to surface run-off rates. The factors noted in Table 6, therefore, are to be considered as tentatively suggested. They are to be used until checked and corrected by further observations. They apply to soil conditions such that the rate of water absorption by the soil at the beginning of a critical period will be about $\frac{2}{3}$ in. per hour, decreasing gradually to about $\frac{1}{3}$ in. per hour at the end of 1 hour.

Notation.—The following notation is used in this paper:

A = Area, in acres, of the district above the point at which storm-water discharge is to be estimated;

T = End of a period of heavy rainfall;

T_m = Time at which the storm-water discharge is a maximum;

B = Beginning of a period of heavy rainfall;

B' = Time, t minutes preceding B;

t = Elapsed time, in minutes, from B to T;

t_m = Elapsed time, in minutes, from B to T_m;

R = Maximum rainfall in one hour; expressed in inches;

I = Maximum rain rate, in inches per hour (or, which is the same, second-feet per acre), for a period of time, t minutes in duration;

I_B = Maximum rain rate, in inches per hour, during t minutes preceding the time, B, during time, B' to B;

u_T = The rate of inflow into storm-water inlets during the time, B to T, expressed in cubic feet per second per acre;

u_B = The rate of inflow into storm-water inlets during the time, B' to B;

C = Volumetric capacity for storm-water (storage capacity) of all conduits, above the point at which discharge is to be estimated, expressed in cubic feet per acre;

C_B = Storm-water contents of the conduits at the time, B, expressed in cubic feet per acre;

C_T = Storm-water contents of the conduits at the time, T, expressed in cubic feet per acre;

$C_T - C_B$ = Therefore the augmentation of the water temporarily stored in the conduits during the time, B to T;

G_B = The water on the ground, on roofs, etc., and in the gutters, flowing toward the storm-water inlets, at the time, B, expressed in cubic feet per acre;

G_T = The water on the ground, etc., flowing toward the storm-water inlets at the time, T;

G_{T+i} = The water on the ground, etc., flowing toward the storm-water inlets, at the end of i minutes, during which the rain intensity is the greatest possible, immediately following a critical period the duration of which is t minutes;

r = The surface run-off rate during the time, B to T, expressed in depth, in inches per hour, or in cubic feet per second per acre;

r' = The surface run-off rate during the time, B' to B, expressed in depth, in inches per hour, or in cubic feet per second per acre;

a = The constant in the equation $r = a\ I$ (and approximately $r' = a\ I_B$). This constant will have different values for different areas. For areas with impervious surface throughout, a is unity;

b = A constant appearing in the formula for rain intensity;

d = The mean discharge of storm-water, in cubic feet per second per acre, at the point for which storm-water discharge is to be estimated, during the time, B to T;

d_T = The storm-water discharge at the time, T, expressed in cubic feet per second per acre;

d_m = The maximum storm-water discharge at the time, T_m, expressed in cubic feet per second per acre;

d_B = The storm-water flow at the time, B, expressed in cubic feet per second per acre;

t_D = The time, in minutes, required for water to flow from the most remote portions of a district to the point at which discharge is to be estimated (including a time allowance, i, for flow to inlets);

i = The time, in minutes, required by water to flow from the outermost portions of the area served by a storm-water inlet to the inlet;

S = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T;

S_m = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T_m;

q = The total depth of rain, in inches, during the time, B to T;

q' = The surface run-off, in inches, during the time, B to T;

e = A constant, appearing in the special case when the limiting rain curve is a parabola.

The Storm-Water Flow.—The maximum quantity of rain, expressed in cubic feet, that may fall in the time, B to T, on one acre, will be $60\ I\ t$. The portion of this water which will reach the storm-water inlets is $60\ r\ t$. This is the surface run-off per unit area, and must be equal to the total flow during the time, B to T, at the point for which discharge is to be estimated, plus the augmentation of the water storage on the ground and in the conduits during the same time.

$$60\ r\ t = 60\ a\ I\ t \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$60\ r\ t = 60\ d\ t + (C_T - C_B) + (G_T - G_B) \quad \ldots\ldots\ldots (2)$$

$$d = r - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots\ldots (3)$$

or $$d = a\ I - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots\ldots (4)$$

But if d is known, and if d_B can be determined, it will be possible to find the value of d_T on the assumption that

$$d = \frac{2}{3} d_T + \frac{1}{3} d_B \ldots\ldots\ldots\ldots\ldots (5)$$

From which $$d_T = \frac{1}{2} (3\ d - d_B) \ldots\ldots\ldots\ldots\ldots (6)$$

The value assumed for d in Equation 5 results from the hypothesis that the hydrograph for the time, B to T, approximates in shape a parabola, tangent to a horizontal line at T and passing through the point B, as shown in Fig. 7.

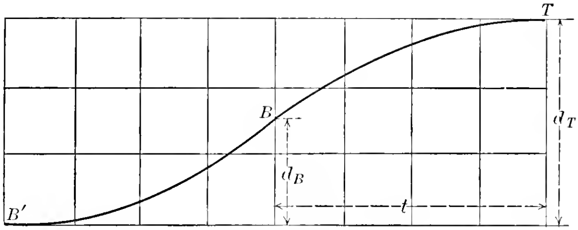


Fig. 7.

But the value of d_B can be approximated on the further assumption that only the rainfall in the t_m minutes preceding the time, B, need be taken into account. For the greatest possible rain intensity preceding B, the value of d_B, without too great error, may be called equal to the mean run-off rate during the t_m minutes preceding B. That is to say:

Because, approximately, $r' = a\ I_B$ .... (7)

Therefore, near enough, $d_B = r' = a\ I_B$ .... (8)

And from Equation 6, $d_T = \frac{1}{2}(3\ d - a\ I_B)$ .... (9)

It appears from the above that the value of d increases with increasing values of C_B. It increases in value, also, for decreasing values of C_T. (See Equation 4.) Therefore the rain which will give d maximum value must be so distributed to the time preceding and immediately following the time, B, that the conduits will be as full of water as possible at the beginning, and at as low a stage as possible at the end, of the critical period. With this fact the type of rain storm becomes known which will make the mean discharge throughout a critical period a maximum.

The rain must evidently fall preceding the time, B, with constantly increasing intensity, and during the time, B to T, with constantly decreasing intensity.

It follows, that the greatest possible rain intensity, at a time instant, immediately preceding B, will be the same as the greatest possible rain intensity at the close of a critical period. Consequently, for the most unfavorable rain conditions, the value of G_B will be equal to G_{T+i} (the inflow time being i minutes).

Equation 3 may now be written

$$d = r - \frac{(C_T - C_B) + (G_T - G_{T+i})}{60t} \quad \text{.........(10)}$$

but $$S = C_T - C_B + G_T - G_{T+i} \quad \text{.....................(11)}$$

therefore $$d = r - \frac{S}{60\,t} \quad \text{..................................(12)}$$

The values of G_T and G_{T+i} can be calculated for any rain curve, as has already been shown. The value of d in Equations 3 and 10 and of d_T in Equation 6 are dependent, not alone upon the intensity of the rain, but also upon the duration of the extreme rain conditions, and upon the volumetric increase of the water in the conduits and on the ground during the critical period.

It may be stated in passing that, for the inlet ends of the conduits, $C_T - C_B = 0$. Therefore, from Equation 10, for the special case in which $d = u$ and $d_T = u_T$

$$u = r - \frac{G_T - G_{T+i}}{60\,t} \quad \text{..................(13)}$$

calling $i = 5$

$$u = r - \frac{G_T - G_{T+5}}{60\,t} \quad \text{..................(14)}$$

and from Equation 6

$$u_T = \frac{1}{2}(3\,u - r') \quad \text{.......................(15)}$$

It may be inferred from Equation 9,

$$d_T = \frac{1}{2}(3\,d - a\,I_B),$$

that the value of d_T will follow closely the value of d; that, therefore, the same type of rain storm which will give the mean discharge during a critical period maximum value will also make the discharge at the time, T, maximum. It is reasonably certain that this is the case, as will be shown later in the presentation of a graphical solution of the problem. This fact is not demonstrated by Equation 9, because C_B is at maximum value when I_B is at maximum value, while d_T increases with the increasing value of C_B and with the decreasing value of I_B. The graphical illustration will show that in all ordinary cases the effect of changing values of C_B upon the value of d_T outweighs the effect of the corresponding changes in the rain rate, I_B. But the introduction of the variable, I_B, into the calculation of d_T from d makes it apparent that the time at which d is maximum is not

necessarily the time at which d_T is maximum. These times, in fact, are not coincident.

The least satisfactory part of the foregoing demonstration lies in the approximation of a value for d_B and in the assumption that the maximum discharge can be computed from the mean discharge during the critical period and the discharge at the beginning of the period.

Nor is it, as a matter of course, strictly correct, except in the case of impervious areas, to assume that the relation of r' to I_B is the same as the relation of r to I. This assumption is probably near enough to the truth for all practical purposes.

The true value of d_T, therefore, may depart to some extent from the value determined, as explained, from d and d_B. The room for such departure will decrease as the values of d_B and d approach each other; it will be greatest when $d_B = 0$; that is, the opportunity for a departure from the correct value is greatest in the case of no rain preceding the critical period. It is quite possible that for this special case some other expression than Equation 9 can be found that will better express the relation between d, d_B, and d_T; but the further investigation of this special case would only be of interest if it were desired to deduce formulas for application to types of rain storms other than those which produce the maximum discharge.

To estimate the values of C_T and C_B, let it be assumed that a first approximation of conduit sizes by any method has established a first value of C. The value of C when the conduits are to carry water other than that due to rain will not, as a matter of course, be the entire volumetric contents of the conduits, but that portion thereof which is available for the temporary storage of rain water. Call t_m equal to t_D. This fixes tentatively the position of T in the time scale. The limiting rain curve furnishes the value of I and of I_B, and, by application of the proper reduction factor, the values of r and of r' are found from $r = a\ I$ and $r' = a\ I_B$.

It is now possible to estimate the maximum value of C_B. This will result, as already explained, if the rain storm is of such a character that, preceding the time, B, there is a rainfall of constantly increasing intensity. See Fig. 7. The flow at the time, B, can now be calculated; but the ratio of this flow to the flow at the time, T, may be, preliminarily, considered the same as the ratio of I_B to I. With the aid of this ratio, on the assumption that the space occu-

pied by water in the conduits is proportional to the flow (the velocities being practically the same for half full and for full or nearly full stages) a first approximation of C_B is possible. The first approximate value of C_T is C. That is, all conduits are assumed to be full at the time, T.

The value of d resulting from this assumption will be too small, because, even under the most unfavorable conditions, C_T will always be somewhat less than C.

The value of d_T follows by the use of Equation 9.

The conduit sizes originally entering into the calculation can now be revised, and the values of C_B and C_T are again determined. It is now desirable to make the estimate of C_T with greater precision than for first approximation purposes. This can readily be done, because, at the time, T, the conduits are full at the point for which storm-water flow is to be estimated and the rate of flow at all upper ends of conduits can be estimated from the rain rate which prevailed in the last i minutes preceding the time, T.

It will ordinarily be found convenient to call

$$C_T = KC \qquad \text{(16)}$$

where K is a coefficient to be ascertained for each area under consideration. The value of K will approach unity for small districts.

From Equations 9 and 10,

$$d = r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}$$

and $$d_T = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}\right) - r'\right]$$

For all cases in which t exceeds 15 min., $G_T - G_{T+5}$ will have so small a value that the term containing this expression may be neglected, therefore $t > 15$

$$(d_T)_{t > 15} = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t}\right) - r'\right] \qquad \text{(19)}$$

or $$(d_T)_{t > 15} = \frac{1}{2}\left[3\left(a\,I - \frac{K\,C - C_B}{60\,t}\right) - a\,I_B\right] \qquad \text{(20)}$$

Numerical Illustrations.—Let it be required to determine the capacity of the outfall sewer for a business district, with impervious surface throughout, in San Francisco, the area of the district being 1 000 acres. Suppose that the district has such shape and such surface slope that it will take water 50 min., including a 5-min. entrance allowance, to flow from the most remote portion thereof to the point

at which the conduit capacity is to be determined. Temporary storage of water on the ground's surface, because $t_D > 15$, may be neglected ($G_T - G_{T+5} = 0$).

The maximum rain rate in San Francisco for 50 min., when $I = \dfrac{3.68}{\dfrac{2\,t}{t+60} + A^{0.4}}$ (see Table 2), is 0.65 in. per hour, or 0.65 sec-ft. per acre. For an impervious surface, $a = 1.00$; therefore $r = I$, and $r' = I_B$.

A first approximation of the storm-water discharge can be made by the method heretofore described as in use in San Francisco.

By this method,

$$d_m = 0.75 \times 0.65 = 0.488 \text{ sec-ft. per acre.}$$

Let it now be assumed that calculations for a number of points in the district have established conduit sizes having a volumetric capacity for storm-water of 1 500 cu. ft. per acre.

Let it further be supposed that calculations already made have established $K = 0.8$.

An examination of the limiting rain curve of the type recommended for use in San Francisco will show that during a period of time, t minutes in duration, immediately preceding or following a critical period of the same duration, the greatest possible rainfall will be, in round numbers, one-half of the rain during the critical period. For approximation purposes, therefore, the quantity of water in the conduits at the beginning of a critical period may be taken at one-half of the amount in the conduits at the close of the period.

Approximately $$C_B = \frac{C_T}{2},$$

but $$C_T = K\,C = 0.8\,C,$$

therefore, $$C_T - C_B = 0.4\,C = 600.$$

The limiting rain curve for San Francisco, based on $I = \dfrac{3.68}{\dfrac{2\,t}{t+60} + t^{0.4}}$, Fig. 6, indicates that in the 50 min. preceding the time, T, there may be a rainfall aggregating 0.531 in.; that is, $I = 0.649$ in. per hour. The rainfall in 50 min. preceding the critical period cannot, as shown by the rain curve and Table 2, exceed 0.261 in., because the total rain in 100 min. cannot exceed 0.792 in.

The rainfall, 0.261 in. in 50 min., represents a rain intensity of 0.313 sec-ft. per acre.

Consequently, $r = a\,I = 0.649$ (a being unity)

$$r' = a\,I_B = 0.313$$

From Equation 19, $d_T = \frac{1}{2}\left[3\left(0.649 - \frac{600}{3\,000}\right) - 0.313\right]$

$$= 0.517 \text{ sec-ft. per acre.}$$

$$A\,d_T = 517 \text{ sec-ft.}$$

As this value is somewhat larger than the first approximation, it may now be necessary to correct the value of C. For the purpose of this illustration, it will be supposed that no correction is necessary.

Other values of t are now to be tried, in order to find that value of t which will give d_T maximum value.

For $t = 40$ min.

$$r = 0.714$$
$$r' = 0.342$$
$$d_T = \frac{1}{2}\left[3\left(0.714 - \frac{600}{2\,400}\right) - 0.342\right]$$
$$= 0.525$$
$$A\,d_T = 525 \text{ sec-ft.}$$

For $t = 30$ min.

$$r = 0.808$$
$$r' = 0.390$$
$$d_T = \frac{1}{2}\left[3\left(0.808 - \frac{600}{1\,800}\right) - 0.390\right]$$
$$= 0.468$$
$$A\,d_T = 468 \text{ sec-ft.}$$

The critical period is evidently about 40 min., and the maximum storm-water discharge, or required conduit capacity, will be about 525 sec-ft.

As a second illustration, let it be required to ascertain the storm-water discharge from a district, 1 000 acres in area, in which only 250 acres of the surface are impervious. Here, again, the time, t_D, is to be approximated from probable conduit gradients. Suppose $t_D = 50$ min.

The value of a, to be used in the equations $r = a\,I$ and $r' = a\,I_B$, will be found in Table 6, that is $a = 0.44$.

Suppose that, by preliminary estimate, $C = 800$, and that $K \doteq 0.8$.

Again accepting the approximate value, $C_B = \frac{C_T}{2}$, makes $C_B = 320$.

For $t = t_D = 50$ min.

$$I = 0.649$$

and $I_B = 0.313$

$$r = 0.44\ I \quad = 0.285$$

$$r' = 0.44\ I_B = 0.138$$

$$d_T = \frac{1}{2}\left[3\left(0.285 - \frac{320}{3\ 000}\right) - 0.138\right]$$

$$= 0.198$$

$$A\ d_T = 198 \text{ sec-ft.}$$

For $t = 40$ min.

$$I = 0.714$$

$$I_B = 0.342$$

$$r = 0.44\ I \quad = 0.315$$

$$r' = 0.44\ I_B = 0.151$$

$$d_T = \frac{1}{2}\left[3\left(0.315 - \frac{320}{2\ 400}\right) - 0.151\right]$$

$$= 0.197$$

$$A\ d_T = 197 \text{ sec-ft.}$$

For $t = 60$ min.

$$I = 0.600$$

$$I_B = 0.291$$

$$r = 0.44\ I \quad = 0.264$$

$$r' = 0.44\ I_B = 0.128$$

$$d_T = \frac{1}{2}\left[3\left(264 - \frac{320}{3\ 600}\right) - 0.128\right]$$

$$= 0.184$$

$$A\ d_T = 184 \text{ sec-ft.}$$

The critical time is evidently about 50 min., and the required capacity about 198 sec-ft.

It must be remembered that the values of the reduction factor a noted in Table 6 are based on a specific rate of water absorption by the soil. They are to be suitably modified whenever, for any district, other rates of water absorption by the soil shall have been determined.

The increase of water contents of the conduits, when conduits are already constructed, should be calculated from the estimated water stages whenever this is practicable, instead of using the above indicated approximation method.

How the value of t_m may be found, when the storage increase during a critical period is known, will be explained later.

Graphical Solution.—The method of estimating the required capacity of storm-water conduits just illustrated becomes of importance, first, because it is based on sound principles and can be made applicable to districts of any shape, character, and size; and second, because it enables the problems to be solved by simple graphical methods, as will now be explained.

With the mass curve of rainfall for extreme conditions of precipitation as a guide, there is to be constructed for any area under consideration a second integral or mass curve representing the maximum amount of surface run-off during the progress of a downpour of the extreme type. The new curve will be the rain curve modified by the appropriate reduction factor, a, to be found in Table 6. The new curve, for convenience, may be called the surface run-off curve.

The surface run-off curve for areas with impervious surface throughout will coincide with the limiting rain curve, because, for the impervious surface, the factor, a, is unity and $r = I$. The San Francisco surface run-off curve for impervious surface, based on rain intensities expressed by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t + 60} + t^{0.4}}$$

(which will closely approximate the rain curve of 1899), is shown in Fig. 9.

The time scale in this diagram indicates minutes subsequent to the time, B. The ordinates represent, in cubic feet per acre, the water which will ultimately reach the storm-water inlets.

The total run-off during any time period is the difference between the ordinates at the beginning and at the end of the period. Some of this water, however, is still flowing across the surface toward the inlets at the end of the period.

The surface run-off curve for an area having a surface which is not impervious throughout can be obtained from the curve for impervious areas by multiplying the ordinates by the proper reduction factors.

Suppose now that the limiting surface run-off curve be platted in a system of co-ordinates with the point, B, at the beginning of the co-ordinates (Fig. 8).

Let T be located on this run-off mass curve so that B to T on the

time scale is the actual critical period for the point of the conduit under consideration.

Make $\qquad T D' = S$

the value of S being $S = C_T - C_B + G_T - G_{T+i}$ . . . . (11)

then $$d = \frac{D' F}{60\ t}$$

The storm-water discharge at the time, B, as already explained, which will make d a maximum, will be the discharge due to a maximum possible fall of rain in the time, t minutes, preceding B. The rainfall in this time must be one of increasing intensity. The run-off curve, therefore, will take the position as shown, from B'' to B.

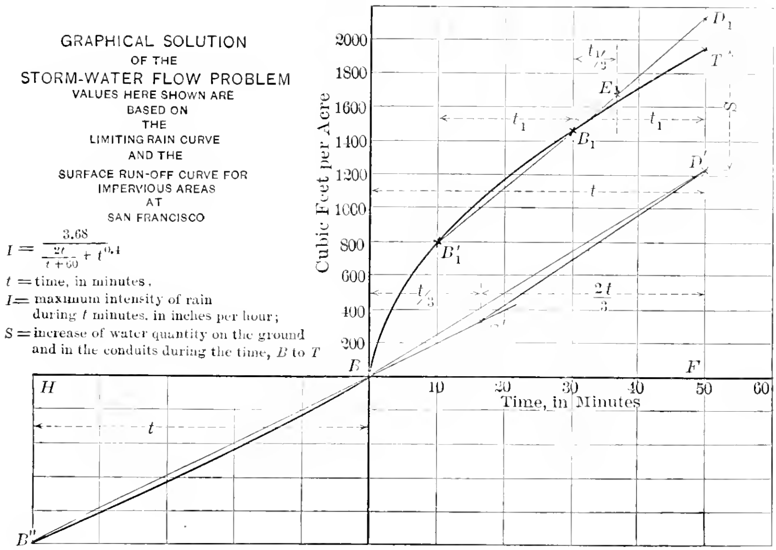


FIG. 8.

The discharge of storm-water at the time, B, may be taken, without material error, as equal to the mean surface run-off rate during the time, B'' to B, provided, of course, that t be not at material variation from t_D. Therefore,

$$d_B = \frac{B''\ H}{60\ t}.$$

The mass curve of storm-water discharge after the time, B, will therefore be represented by a curve to which the line, $B''\ B$, is tangent at B, and which will gradually rise to the point, D'. The inclination

of the tangent to this curve at the point, D', will represent the maximum flow at the time, T.

The shape of this curve will probably be such that its change of direction is somewhat more rapid at the time, B, at which the downpour is most intense, than at the time, T, at which the flat top of the flood wave in the conduit has reached the point for which discharge is being estimated. It will be reasonable to assume, if $B''\ B$ be

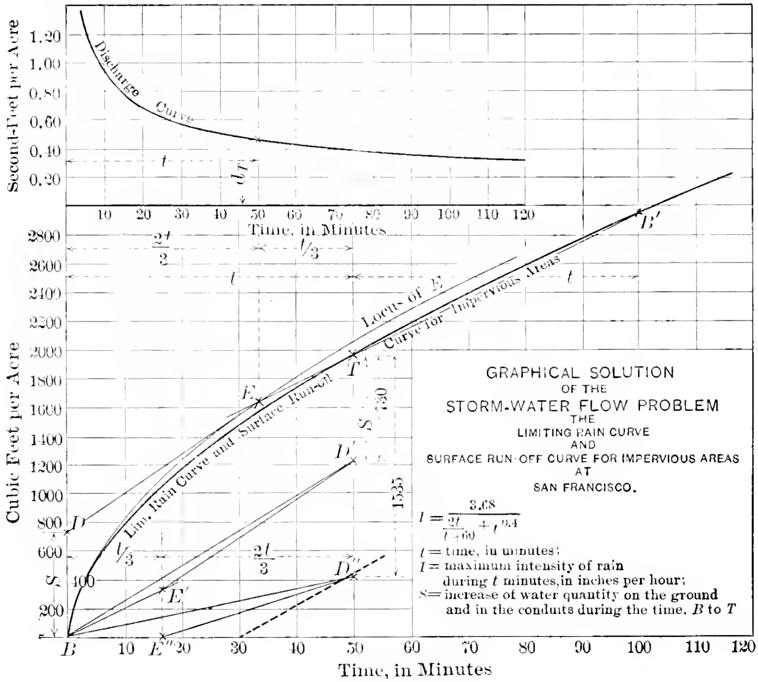


FIG. 9.

prolonged to the point, E', on the time ordinate, one-third of the way from B to T, that the line, $E'\ D'$, will be a close approximation to the inclination of the curve at the point, D'. This is in conformity with the assumption heretofore made that

$$d = \frac{2\ d_T}{3} + \frac{d_B}{3} \quad \dots\dots\dots\dots\dots\dots\dots (5)$$

By assuming a new position for the point, T, with corresponding modification of the curve, B'' to B, a new position of the point, E', will be found. It thus becomes possible, by determining successive locations of the points, E' and D', to find the particular value of t

which will give the line, E' D', its greatest inclination; which will, in other words, for the known value of S, give d_T maximum value.

The same result may be obtained in a much simpler way. Suppose the value of S to be scaled off upward from B in the diagram, Fig. 9, establishing the point, D. Let B' be the point on the run-off mass curve for a time period equal to 2 t. Then the line, B' T, of this diagram will be parallel to the line, B'' B, of the diagram, Fig. 7. If this line, B' T, prolonged cuts the time ordinate lying two-thirds of the way from B to T at E, then D E will be parallel to the line, E' D', of the diagram, Fig. 8, and the inclination of the line, D E, will give the desired value of d_T.

For every possible location of T a new position of E will be found. The successive positions of E will lie upon a curve which, as will readily be seen, is always the same no matter what the value of S may be. Therefore, to find the maximum value of d_T, it will only be necessary to draw a tangent from the point, D, Fig. 9, to the E-curve.

The point of tangency thus found will be distant from B on the time scale two-thirds of the time, t_m; and the inclination of the tangent will be the greatest possible value of d_T.

For the limiting rain curve of San Francisco, based upon

$$I = \frac{3.68}{\frac{2\,t}{t + 60} + t^{0.4}}$$

and impervious surfaces ($a = 1$), the resulting locus of the point, E, or the E-curve, is shown in Fig. 9.

Suppose, now, that a value of C_T — C_B has been approximated. Add to this G_T — G_{T+5}, obtained from Table 5, on the supposition that $i = 5$ and that t_m will be about equal to t_D

Scale upward from B the value of S thus obtained, and draw D E tangent to the E-curve at E.

Then $d_T =$ inclination of line, D E.

and $t_m =$ one and one-half times the scaled time from B to E.

It will be convenient to complete the diagram by platting to suitable scale the differentials of the E-curve. A third curve will thus be obtained which may be called the discharge curve. This curve will give at once the value of d_m for any value of t_m.

By calling $t_m = t_D$, a quick first approximation of the value d_m can thus be made.

It follows from the preceding demonstrations that for every mass

curve of surface run-off, on the hypotheses as made, there is a definite inter-relation between the values of S_m, t_m, and d_m. This inter-relation can be determined from the diagram, and can be tabulated.

Ordinarily, the fact most readily ascertainable for any district is the value of t_D, which (in the case of urban storm-water flow problems) may be accepted as a first approximation of the value of t_m. In tabulating the information obtained by diagram it will, therefore, be convenient to note regular time intervals in the first column.

The values noted in Columns 2, 3, and 4 of Table 7 are based on the limiting rain curve for San Francisco as determined by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

and are applicable to areas with impervious surface throughout.

The corresponding columns in Table 8 contain values based on the same limiting rain curve with reductions due to water absorption by soil when only 25% of the surface is impervious. (For the reduction factors used see Table 6.)

TABLE 7.—Storm-Water Flow in Its Relation to the Duration of the Critical Time Period, and to the Augmentation of the Water Quantity Temporarily Stored in the Conduits.

For areas with impervious surface throughout. For San Francisco.

$$\text{Based on } I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}.$$

Duration of critical period, t_m, in minutes.	Augmentation of storage in the conduits, $C_T - C_B$, in cubic feet per acre.	By Diagram. Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	By Diagram. Discharge, d_m, in second-feet per acre.	Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
5	5	200	1.25	1.34
10	270	300	0.93	0.99
15	365	385	0.78	0.82
20	440	450	0.69	0.71
30	545	555	0.57	0.60
40	635	640	0.51	0.54
50	725	730	0.46	0.49
60	800	800	0.43	0.45
70	870	870	0.40	0.43
80	930	930	0.38	0.41
90	985	985	0.36	0.38
120	1 120	1 120	0.32	0.34

TABLE 8.—STORM-WATER FLOW, IN ITS RELATION TO THE DURATION OF THE CRITICAL PERIOD AND TO THE AUGMENTATION DURING SUCH PERIOD OF THE WATER QUANTITY TEMPORARILY STORED IN THE CONDUITS.

For areas with 25% of the surface impervious. For San Francisco.

$$\text{Based on } I = \frac{3.68}{\frac{2\,t}{t + 60} + t^{0.4}}$$

Duration of critical period, t_m, in minutes.	Augmentation of the storage in the conduits, $C_T - C_B$, in cubic feet per acre.	By Diagram.		Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
		Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	Discharge, d_m, in second-feet per acre.	
5	5	140	0.89	0.63
10	170	190	0.60	0.46
15	205	220	0.45	0.39
20	230	240	0.37	0.34
30	265	270	0.28	0.28
40	290	295	0.24	0.25
50	320	320	0.20	0.23
60	345	345	0.19	0.21
90	375	375	0.14	0.18
120	415	415	0.12	0.16

Referring again to Fig. 7, it remains to be shown that no rain condition preceding the time, B, can be assumed which will give the line, $E'\,D'$, greater inclination than resulting from that which has already been declared to be the most unfavorable.

Assume the extreme case of no rain preceding the time, B. In this case there will be no storm-water in the conduits and no water on the ground at the time, B. Consequently, $C_B = 0$, and $G_B = 0$.

From Equation 11, $S = C_T + G_T$;

but, as has already been shown, the maximum value of C_B for extreme rain intensity preceding the time, B, is about one-half of C_T; therefore, approximately, for this condition,

$$C_T = 2\,(C_T - C_B),$$

where C_B represents volumetric water contents of the conduits due to maximum rainfall conditions prior to the time, B.

For the special case shown in the diagram,

for $t = 50$,	$C_T - C_B =$	730,
therefore	$C_T =$	1 460.
From Table 5,	$G_T =$	75
consequently	$S =$	1 535.

If the critical time were the same for the condition of no rain preceding the time, B, as for the condition of heavy rain preceding B, then it would be proper to scale this value of S downward from T in Fig. 9, and it would at once be apparent, because the inclination of $E'' D''$ is less than the inclination of $E' D'$, that the most unfavorable assumption is that of heavy rain preceding the critical period. But the durations of the critical periods are not the same in the two cases. On the same hypothesis on which Equation 5 is based, the critical period for the case of no preceding rain is of nearly twice the duration as for the case of heavy preceding rain. For the special values above noted, one-third of the critical period would be about 30 min., and the corresponding value of d'_m would be the inclination of the broken line (Fig. 9) rising from the 30-min. point of the base line.

This, also, is less than that resulting from the originally assumed unfavorable conditions. The same result can be shown for any other value of t and for any other rain intensities preceding the time, B, less than those originally assumed as the least favorable.

The conclusion relating to the most unfavorable distribution of rain heretofore announced appears therefore to be correct. The rain rates that should be assumed as prevailing before the beginning of the critical period are the greatest rates that are possible, the rain being such that it falls with gradually increasing intensity until the time, B.

The comparison of the results by the new method of computation with those obtained by the method of 1899, as shown in Table 7, indicate reasonably close agreement for areas with impervious surface.

The comparison in Table 8 for areas which are only in small part impervious indicates a greater range of unit values for the new method than result from the older method of calculation. The method of 1899 probably gives too small values for small areas which are only partly impervious.

It may be repeated that water quantities in the conduits at the times, B and T, can ordinarily be calculated with a sufficient degree of approximation. At the time, T, the main conduit is flowing at capacity at the point for which storm-water flow is to be estimated, and the inlets on all the conduit branches are at the same time receiving water at the run-off rate which corresponds to the last few minutes of

the critical time period. It is possible, therefore, to determine to what proportion of their capacity the conduits are charged at their inlet ends. With this information for the upper ends of conduits, and the knowledge that the conduits are full near the point under consideration, a close approximation of the water stage at the time, T, is possible, and C_T may be calculated.

By a similar line of reasoning, the stage of water in the conduits can be determined for the time, B, and the value of C_B can also be calculated.

But for first approximation purposes it will be well to establish the values of K for increasing areas. Then call the ratio of the water stored in the conduits at the time, B, to the water stored at the time, T, equal to the ratio of the mean run-off during the t_p minutes preceding B to the mean run-off in the t_p minutes after B. For San Francisco this ratio, as already stated, may be called as one to two.

That is, $$C_B = \frac{C_T}{2};$$

but $$C_T = K\,C,$$

therefore $$C_B = \frac{K\,C}{2},$$

and $$C_T - C_B = \frac{K\,C}{2}.$$

To the value of $C_T - C_B$ thus or otherwise determined add the value of $G_T - G_{T+i}$, as found in Column 7 of Table 5 for $i = 5$. The sum will be the value of S to be used, as already explained.

When this method of determining storm-water flow is applied to a conduit system already constructed, it may happen that, owing to excessive or deficient storage capacity in the conduits, there will be wide divergence in the values of t_m and t_p.

The value of t_m may be considerably less than t_p if the conduits are too small, if they are surcharged at critical times; and it may be materially larger than t_p if along the line of the conduits there is unusual reservoir space to be filled before the maximum stage is attained. It will be nearly twice as great as t_p for the special case of no rain preceding the critical period.

The graphical method of determining the storm-water flow is applicable to any limiting mass curve of surface run-off. It is applicable therefore to any such curve obtained by applying a reduc-

tion factor to the run-off rates from areas with impervious surfaces.

But when the problem is presented of finding the storm-water flow for districts having surfaces which are not impervious throughout, it will not be necessary to construct new curves. The original curve for impervious conditions throughout may be used to solve any such problem.

When the value of $S = C_T - C_B + G_T - G_{T+i}$ has been determined for an area of any surface character, divide this value by the factor, a (to be found in Table 6), and proceed with the modified value of S thus obtained ($S' = \frac{S}{a}$), as though the area had an impervious surface throughout. There will thus be obtained a modified value of d_T, which may be called d'_T, such that

$$d'_T = \frac{d_T}{a},$$

$$\text{therefore,} \quad d_T = a\, d'_T.$$

That is to say, the value thus obtained from the curves or from the tables is to be multiplied by the same reduction factor before used, to give the value of d_T.

New Formula and Discharge Tables.—A study of the limiting rain curve of 1899 for San Francisco has led to the adoption of a formula, as already stated, which, it is believed, will be found generally applicable for the determination of rain intensities for all periods of time that come under consideration in urban run-off problems.

In its general form the formula may be written

$$I = \frac{b}{\dfrac{2\,t}{t + 60} + t^{0.4}} \qquad \text{(21)}$$

Here b is a constant for any locality, and t is the duration of the rain, in minutes.

This formula gives the value of I, in inches per hour, or, which is the same thing, in cubic feet per second per acre.

For San Francisco, $b = 3.68$ will give values closely agreeing with those determined in 1899. (See Table 3.)

For Atlantic Slope conditions, ordinarily, $b = 8$ to 12.

Wherever the maximum rain intensity for 1 hour is known, the value of b can be determined. It will generally be well, however, provided data are adequate, to base the value of b upon determinations from the maximum rain intensities for 10 min., 30 min., and 1 hour. In most cases the use of an average value from these three determinations should satisfy every requirement.

The rain intensities resulting from $b = 3.68$, 4, 8, and 12, in this formula, have already been noted in Table 3.

Whether the expression recommended, or any other expression in the general form

$$I = c\, f\,(t) \quad \ldots\ldots (22)$$

be used in estimating the maximum rain intensity during the time, t, the fact is noteworthy that for all possible values of c or of b in the other case, the corresponding ordinates of the resulting mass curves will be proportional to the values of c or b that have been used.

This fact makes it possible to establish by a single diagram (for impervious surface conditions), base values of storage augmentation and of storm-water flow, being for the special case of $b = 1$ or of $c = 1$.

From such base values the storage and discharge for other values of b or of c, and also for any condition of the surface, can then be found by multiplication with the factors, a, b or a c, as the case may be.

Table 9 contains the base values of S_m, d_m, and $(C_T - C_B)$, that is $(S_m)_{\substack{a=1\\b=1}}$, $(d_m)_{\substack{a=1\\b=1}}$, and $(C_T - C_B)_{\substack{a=1\\b=1}}$, for the type of rain curve herein recommended, as determined by diagram.

The values in the last column of Table 9 conform closely to a curve herein recommended, as determined by diagram.

$$y = \frac{0.71}{\frac{2\,t_m}{t_m + 60} + t_m^{\,0.4}} \quad \ldots\ldots (23)$$

This is the value of d_m for the special case in which $a = 1$ and $b = 1$. For any value of a and of b, therefore,

$$d_m = \frac{0.71\, a\, b}{\frac{2\,t_m}{t_m + 60} + t_m^{\,0.4}} = 0.71\, a\, I \quad \ldots\ldots (24)$$

As it frequently will be desirable to compute the discharge directly from the maximum rainfall in 1 hour, the value of b may be found in terms of R and substituted.

For $t = 60$, there will be $I = R$.

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{\,0.4}} \quad \ldots\ldots (21)$$

$$R = \frac{b}{6.14} \quad \ldots\ldots (25)$$

$$b = 6.14\, R \quad \ldots\ldots (26)$$

Therefore, from Equation 24,

$$d_m = \frac{4.36\, a\, R}{\dfrac{2\, t_m}{t_m + 60} + t_m^{\;0.4}} = 4.36\, a\, R \left(\frac{I}{b}\right) \quad \ldots\ldots\ldots\ldots (27)$$

TABLE 9.—BASE VALUES OF d_m AND $C_T - C_B$;
WHEN $a = 1$ AND $b = 1$:

IN THE FORMULA FOR SURFACE RUN-OFF:

$$r = a\, I = \frac{a\, b}{\dfrac{2\, t}{t + 60} + t^{0.4}}$$

All values determined by diagram.
(Subject to correction.)

Duration of the critical period, t_m, in minutes.	Total storage increase, (S_m) $a = 1$ $b = 1$, in cubic feet per acre.	Increase of storage in the conduits, ($C_T - C_B$) $a = 1$ $b = 1$, in cubic feet per acre.	Discharge, (d_m) $a = 1$ $b = 1$, in second-feet per acre.
5	54	0	0.340
10	82	73	0.252
15	105	100	0.212
20	122	120	0.187
30	151	150	0.156
40	174	170	0.139
50	198	197	0.125
60	218	218	0.116
90	268	268	0.097
120	305	305	0.087

In the practical application of this formula, a should be considered constant for each area, as already explained. Its value is directly dependent upon the rate of water absorption by exposed soil surfaces during heavy rains. The value of a for any value of b and any value of t, therefore, can be determined by experiment. For surfaces impervious throughout, the value of a is unity.

Some other values of a preliminarily suggested for use in this formula are to be found in Table 6.

The foregoing discharge formula will be directly applicable to the determination of the maximum discharge when, for any district under consideration, the value of t which will make d_T a maximum is known.

As a first approximation, without great error, t_m may be called equal to t_p. Consequently, a first approximation of d_T by the formula is possible whenever the time can be estimated that it will

take water to flow from the most remote portions of a district to the point at which discharge is being estimated.

It can readily be seen, when

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{0.4}} \qquad (21)$$

that $$(G_T - G_{T+i}) = a\,b\,[G_T - G_{T+i}]_{\substack{a=1\\b=1}} \qquad (28)$$

and that $$(C_T - C_B) = a\,b\,[C_T - C_B]_{\substack{a=1\\b=1}} \qquad (29)$$

consequently, also, $$S = a\,b\,(S)_{\substack{a=1\\b=1}} \qquad (30)$$

From the values given for $(S_m)_{\substack{a\ 1\\b\ 1}}$ in Table 9, for the special case of $i = 5$, the relation between t_m and $(S_m)_{\substack{a\ 1\\b\ 1}}$ can be reduced to formula. It will be found that, with a sufficient degree of accuracy for all values of t_m which are likely to arise in connection with urban problems,

$$(S_m)_{\substack{a\ 1\\b\ 1}} = 27.5\,\sqrt{t_m} \qquad (31)$$

herefore $$S_m = 27.5\,a\,b\,\sqrt{t_m} \qquad (32)$$

and $$t_m = \left(\frac{S_m}{27.5\,a\,b}\right)^2 \qquad (33)$$

From Equations 11 and 32,

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - (G_T - G_{T+i}) \qquad (34)$$

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - a\,b\,(G_T - G_{T+i})_{\substack{a\ 1\\b\ 1}} \qquad (35)$$

and when $i = 5$,

$$C_T - C_B = a\,b\left[27.5\,\sqrt{t_m} - (G_T - G_{T+5})_{\substack{a\ 1\\b\ 1}}\right] \qquad (36)$$

It has already been shown that

$$b = 6.14\,R \qquad (26)$$

Consequently, from Equation 32,

$$S_m = 169\,a\,R\,\sqrt{t_m} \qquad (37)$$

and from Equation 36,

$$C_T - C_B = a\,R\left[169\,\sqrt{t_m} - 6.14\,(G_T - G_{T+5})_{\substack{a\ 1\\b\ 1}}\right] \qquad (38)$$

Likewise, from Equation 37,

$$t_m = \left(\frac{S_m}{169\,a\,R}\right)^2 \qquad (39)$$

and, convenient for purposes of preliminary approximation, from Equations 27 and 37,

$$d_m = \frac{S_m}{38.8 \sqrt{t_m}} \left(\frac{1}{\frac{2\, t_m}{t_m + 60} + t_m^{0.4}} \right) \qquad \ldots\ldots\ldots\ldots (40)$$

TABLE 10.—Storm-Water Flow.

For $b = 4$, in the Formula, $I = \dfrac{b}{\frac{2\, t}{t + 60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\, a\, b}{\frac{2\, t_m}{t_m + 60} + t_m^{0.4}} = \frac{4.36\, a\, R}{\frac{2\, t_m}{t_m + 60} + t^{0.4}}$$

$$S_m = 27.5 \sqrt{t_m}$$

$$\text{and } C_T - C_B = S_m - (G_T - G_{T+5})$$

$$b = 4 \text{ and } R = 0.65.$$

Duration of critical period, t_m, in minutes.	All impervious.		75% impervious.		50% impervious.		25% impervious.		None impervious.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre	d_m second feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	35	1.38	35	1.26	30	1.13	30	1.01	25	0.88
10	310	1.02	280	0.91	250	0.79	210	0.67	165	0.55
15	400	0.85	350	0.74	295	0.63	240	0.51	190	0.40
20	480	0.74	410	0.63	345	0.53	255	0.42	205	0.32
30	600	0.62	500	0.53	410	0.43	315	0.33	220	0.23
40	690	0.55	580	0.46	460	0.37	350	0.28	235	0.185
50	780	0.50	650	0.42	510	0.33	380	0.25	240	0.155
60	850	0.46	700	0.38	550	0.30	400	0.215	245	0.135
90	1 050	0.39	860	0.32	660	0.245	460	0.170	260	0.098
120	1 210	0.35	980	0.28	740	0.215	510	0.145	275	0.080

When, therefore, the increase of water storage during a critical time period can be determined, the duration of this period can be found from Equations 33 or 39, and d_m can be estimated by formula, Equations 24 or 29; or the value of d_m can be taken from Tables 10, 11, and 12, or from the general table, Table 13. Or, the value of d_m may be estimated directly from the value of S_m and an approximate value of t_m by the use of Equation 40. In this last case the estimated discharge is to be treated as an approximation until the value of t_m has been verified.

TABLE 11.—Storm-Water Flow.

For $b = 8$, in the Formula, $I = \dfrac{b}{\dfrac{2\,t}{t + 60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\,a\,b}{\dfrac{2\,t_m}{t_m + 60} + t_m^{\;0.4}} = \frac{4.36\,a\,R}{\dfrac{2\,t_m}{t_m + 60} + t^{\,0.4}}$$

$$S_m = 27.5\,a\,b\,\sqrt{t_m}$$

$$\text{and } C_T - C_B = S_m - (G - G_{T\,+\,5})$$

$$b = 8 \text{ and } R = 1.30.$$

Duration of critical period, t_m, in minutes.	All impervious.		75% impervious.		50% impervious.		25% impervious.		None impervious.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	70	2.77	70	2.65	65	2.52	65	2.40	60	2.27
10	620	2.03	590	1.91	550	1.79	520	1.68	480	1.56
15	810	1.70	760	1.58	710	1.48	660	1.37	600	1.26
20	960	1.49	900	1.38	830	1.28	760	1.18	690	1.07
30	1 190	1.24	1 100	1.14	1 010	1.04	920	0.94	820	0.86
40	1 380	1.10	1 270	1.01	1 150	0.91	1 040	0.82	920	0.74
50	1 550	1.00	1 440	0.91	1 330	0.82	1 170	0.74	1 010	0.65
60	1 710	0.93	1 560	0.85	1 410	0.76	1 260	0.68	1 100	0.60
90	2 090	0.78	1 900	0.71	1 700	0.64	1 510	0.56	1 310	0.49
120	2 420	0.70	2 190	0.64	1 960	0.57	1 730	0.50	1 500	0.43

Tables 10, 11, 12, and 13 have been prepared by the use of the foregoing formulas, all based on

$$I = \frac{b}{\dfrac{2\,t}{t + 60} + t^{\,0.4}}$$

$$i = 5$$

and an absorption of water by the soil at the rate of about $\frac{2}{3}$ in. per hour at the beginning of a critical period, decreasing gradually to a rate of $\frac{1}{3}$ in. per hour at the end of 1 hour.

In the case of conduits already constructed, neither the value of $C_T - C_B$, nor the value of d_m, taken from the tables for $t_m = t_p$, should be considered final.

To facilitate the use of the formula and of the reduction factors, a, Table 13 has been prepared. In this table the value of

$$\frac{0.71}{\dfrac{2\,t_m}{t_m + 60} + t_m^{\;0.4}}$$

appears in Column 3, and the value of

$$\left[27.5\sqrt{t_m} - (G_T - G_{T+5})\right]_{\substack{a=1\\b=1}}$$

in Column 2. The value of $a\ b$ is given in Columns 4 to 18.

To find the discharge, by this table, multiply the value found in Column 3 by the appropriate factor taken from Columns 4 to 18; and to find the increase of the water stored in the conduits, multiply the values found in Column 2 by the same factor.

The table, as already stated, is based on the assumption that $i = 5$; and that, during the time, t_m, the absorption of water by exposed soil surface is at rates decreasing gradually from about $\frac{2}{3}$ in. per hour to $\frac{1}{3}$ in. per hour at the end of 1 hour.

TABLE 12.—Storm-Water Flow.

For $b = 12$, in the Formula, $I = \dfrac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\ a\ b}{\dfrac{2\,t_m}{t_m+60} + t_m^{\ 0.4}} = \frac{4.36\ a\ R}{\dfrac{2\,t_m}{t_m+60} + t_m^{\ 0.4}}$$

$$S_m = 27.5\ a\ b\sqrt{t_m}$$

$$\text{and } C_T - C_B = S_m - (G_T - G_{T+5})$$

$$b = 12 \text{ and } R = 1.95.$$

Duration of critical period, t_m, in minutes.	All impervious.		75% impervious.		50% impervious.		25% impervious.		None impervious.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	100	4.15	100	4.03	95	3.90	90	3.78	90	3.65
10	930	3.05	900	2.95	860	2.84	820	2.72	780	2.60
15	1 210	2.54	1 160	2.44	1 110	2.34	1 060	2.21	1 010	2.11
20	1 440	2.23	1 380	2.14	1 310	2.03	1 240	1.92	1 170	1.80
30	1 790	1.86	1 700	1.73	1 610	1.68	1 520	1.58	1 420	1.47
40	2 070	1.66	1 960	1.57	1 810	1.48	1 730	1.39	1 610	1.29
50	2 330	1.50	2 200	1.41	2 060	1.32	1 930	1.25	1 790	1.15
60	2 560	1.39	2 410	1.31	2 260	1.22	2 110	1.14	1 950	1.06
90	3 140	1.18	2 950	1.11	2 750	1.04	2 550	0.95	2 350	0.88
120	3 620	1.04	3 390	0.98	3 150	0.90	2 910	0.83	2 670	0.77

TABLE 13.—VALUES OF $a\,b$,

WHEN $I = \dfrac{b}{\dfrac{2\,t}{t + 60} + t^{0.4}}$ AND $d_m = a\,b\,\dfrac{0.71}{\dfrac{2\,t_m}{t_m + 60} + t_m^{0.4}}$

$S_m = a\,b\,(27.5\,\sqrt{t_m})$ and $C_T - C_B = a\,b\,[C_T - C_B]_{\substack{a=1\\b=1}}$.

Duration of critical period, t_m, in minutes.	BASE VALUES, for $b=1$ and $a=1$.		VALUES OF $a\,b$:															
			For $b=4$ and $R=0.65$.					For $b=8$ and $R=1.30$.					For $b=12$ and $R=1.95$.				
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
5	[illegible]	0.347	4.00	3.65	3.30	2.90	2.60	8.00	7.70	7.30	6.95	6.55	12.00	11.60	11.30	10.90	10.60
10	78	0.254	4.00	3.55	3.10	2.65	2.15	8.00	7.50	7.05	6.65	6.15	12.00	11.55	11.15	10.70	10.20
15	107	0.212	4.00	3.50	2.95	2.40	1.90	8.00	7.45	6.95	6.50	5.90	12.00	11.50	10.95	10.45	9.95
20	125	0.186	4.00	3.45	2.90	2.25	1.70	8.00	7.40	6.90	6.30	5.75	12.00	11.45	10.90	10.30	9.70
30	150	0.156	4.00	3.40	2.75	2.10	1.50	8.00	7.35	6.75	6.10	5.50	12.00	11.40	10.80	10.20	9.50
40	170	0.138	4.00	3.35	2.70	2.00	1.35	8.00	7.35	6.65	6.00	5.30	12.00	11.35	10.70	10.10	9.35
50	195	0.125	4.00	3.30	2.65	1.95	1.25	8.00	7.30	6.60	5.90	5.20	12.00	11.30	10.60	9.95	9.20
60	215	0.116	4.00	3.30	2.60	1.90	1.15	8.00	7.30	6.55	5.80	5.10	12.00	11.30	10.55	9.80	9.10
90	260	0.098	4.00	3.25	2.50	1.75	1.00	8.00	7.25	6.50	5.75	5.00	12.00	11.30	10.50	9.70	9.00
120	300	0.087	4.00	3.25	2.50	1.70	0.90	8.00	7.25	6.50	5.75	4.95	12.00	11.30	10.50	9.60	8.90

The type of the formula for rain intensity, as here recommended, is the direct outcome of an attempt to make the resulting values follow closely the San Francisco limiting rain curve for all values of t up to 24 hours.

The parabola, as a limiting curve, was tried, but, for higher values of t, departed too far from the curve of 1899 to be considered entirely satisfactory.

When all time periods that come into consideration are less than 2 hours, as will be the case in nearly all urban problems, a parabola can frequently be found which will closely approximate the limiting rain curve, and may be used in its stead.

In such case let the equation of the limiting rain curve be

$$q = e\sqrt{t} \text{ (total rainfall in inches)} \qquad (41)$$

The equation of the surface run-off curve will then be

$$q' = a\, e\sqrt{t} \qquad (42)$$

The value of a, as has already been explained, is dependent upon the amount of impervious surface in any area, and varies, too, with the duration of the critical period. For impervious areas, a is unity.

For $t = 60$, the value of q will be the maximum rainfall in 1 hour, therefore, from Equation 41,

$$R = e\sqrt{60} \qquad (43)$$

$$e = \frac{R}{7.746} \qquad (44)$$

$$e = 0.129\, R \qquad (45)$$

$$\text{therefore } q' = 0.129\, a\, R\sqrt{t} \qquad (46)$$

The maximum intensity of the rainfall for any number of minutes can be found from

$$I = \frac{60\, q}{t} = \frac{60\, e}{\sqrt{t}} \qquad (47)$$

$$\text{and } I = \frac{7.75\, R}{\sqrt{t}} \qquad (48)$$

By reference to Fig. 10 it can be seen that for a curve the equation for which is

$$q' = a\, e\sqrt{t} \qquad (42)$$

the equation of the line, $B'\,T$, can be written

$$y = \frac{0.414\, a\, e}{\sqrt{t}}\, x + 0.586\, a\, e\sqrt{t} \qquad (49)$$

For $x = \frac{2\,t}{3}$, this equation will give the ordinate of the point, E.

The locus of the point, E, will be expressed by

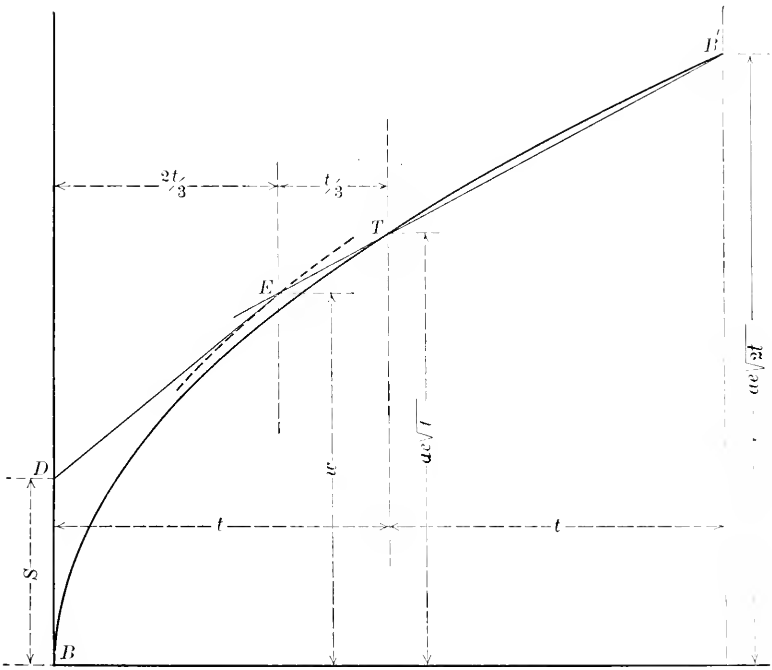
$$w = \frac{0.828\ a\ e\ t}{3\sqrt{t}} + 0.586\ a\ e\sqrt{t} \quad \ldots\ldots\ldots\ldots (50)$$

$$w = 0.862\ a\ e\sqrt{t} \quad \ldots\ldots\ldots\ldots (51)$$

$$\text{or}\ w = 1.056\ a\ e\sqrt{x} \quad \ldots\ldots\ldots\ldots (52)$$

Therefore the tangent to the E-curve at the point, E,

$$\frac{d\,w}{d\,x} = \frac{0.528\ a\ e}{\sqrt{x}} = \frac{0.647\ a\ e}{\sqrt{t}} \quad \ldots\ldots\ldots\ldots (53)$$



WHEN THE LIMITING RAIN CURVE
IS A PARABOLA

FIG. 10.

The value of $\frac{d\,w}{d\,x}$ as thus found will be inches in depth in 1 min.

Therefore d_m, in inches in 1 hour, or in second-feet per acre,

$$d_m = 60\ \frac{0.647\ a\ e}{\sqrt{t}} = \frac{38.8\ a\ e}{\sqrt{t_m}} \quad \ldots\ldots\ldots\ldots (54)$$

And, by substitution,

$$d_m = \frac{5.00\ a\ R}{\sqrt{t_m}}\ \text{cu. ft. per sec. per acre} \quad \ldots\ldots\ldots\ldots (55)$$

The value of S_m, expressed in inches of depth, will be

$$w = \frac{2\, t_m}{3} \left(\frac{0.647\, a\, e}{\sqrt{t_m}} \right)$$

from which it follows:

$$S_m = 3\,630\, [0.862\, a\, e \sqrt{t_m} - 0.431\, a\, e \sqrt{t}\,] \ldots\ldots\ldots\ldots (56)$$

$$S_m = 1\,565\, a\, e \sqrt{t_m} \text{ cu. ft. per acre} \ldots\ldots\ldots\ldots (57)$$

From Equation 57, $$t_m = \left(\frac{S_m}{1\,565\, a\, e} \right)^2 \ldots\ldots\ldots\ldots (58)$$

By substitution from Equation 45,

$$t_m = \left(\frac{S_m}{202\, a\, R} \right)^2 \ldots\ldots\ldots\ldots (59)$$

and $$S_m = 202\, a\, R \sqrt{t_m} \ldots\ldots\ldots\ldots (60)$$

Finally, from Equations 54 and 58,

$$d_m = \frac{60\,700\, a\, e^2}{S_m} \ldots\ldots\ldots\ldots (61)$$

and from Equations 55 and 60,

$$d_m = \frac{S_m}{40.4\, t_m} \ldots\ldots\ldots\ldots (62)$$

The value of a, for use in the foregoing equations, may be taken from Table 6, for the soil absorption conditions on which that table is based.

For San Francisco, the value of e to be used in these equations is 0.0775, the value of R, the greatest possible rainfall in 1 hour, being 0.60 in.

For the rainfall conditions prevailing on the Atlantic Slope, the value of e will ordinarily lie between 0.15 and 0.25.

In order that the importance of using good judgment in the construction of the limiting rain curve may become apparent, Tables 14 and 15 have been prepared.

In Table 14 corresponding values of S_m and d_m are given for the various values of t_m, estimated in the one case by the use of formulas based on limiting rain curves of the San Francisco type, and in the other by the use of formulas based on the parabola type of curve. This table is of general application.

In Table 15 the comparison is made between corresponding values of t_m and d_m for selected values of S_m; but in this case a specific value of R was assumed: $R = 0.60$. The figures on the same lines are not those that would result from an adaptation of curves of the two

types to the rain records, because properly selected curves may not have coincident values of R.

For the San Francisco type of curves:

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{0.4}}$$

$$d_m = \frac{4.36\;a\;R}{\frac{2\,t_m}{t_m + 60} + t_m^{\;0.4}}$$

and
$$S_m = 169\;a\;R\sqrt{t_m}$$

For the parabola:

$$I = \frac{60\;c}{\sqrt{t}}$$

$$d_m = \frac{5\;a\;R}{\sqrt{t_m}} = \frac{S}{40.4\;t}$$

$$S_m = 202\;a\;R\sqrt{t_m}$$

TABLE 14.—Comparative Values of Storm-Water Flow Estimated for Two Types of the Limiting Rain Curve.

For impervious areas throughout.

$a = 1$

Duration of the critical period, t_m, in minutes.	For a Limiting Rain Curve of the San Francisco Type.		When the Limiting Rain Curve is a Parabola.	
	S_m, cubic feet per acre.	d_m, second-feet per acre.	S_m, cubic feet per acre.	d_m, second-feet per acre.
5	378 R	2.13 R	452 R	2.24 R
10	535 R	1.56 R	640 R	1.58 R
15	655 R	1.30 R	780 R	1.29 R
20	755 R	1.14 R	905 R	1.12 R
30	930 R	0.96 R	1 010 R	0.91 R
40	1 070 R	0.85 R	1 280 R	0.79 R
50	1 200 R	0.77 R	1 430 R	0.71 R
60	1 310 R	0.71 R	1 570 R	0.65 R
90	1 600 R	0.60 R	1 920 R	0.53 R
120	1 850 R	0.54 R	2 220 R	0.46 R

Application of the New Method to Special Cases.—Should it be desired to know the extent to which a branch conduit contributes to the discharge of a main conduit at the time, T, two procedures are possible. The discharge estimate may be made for the main conduit for points just above and just below the entrance of the sub-main. The difference between the two results will be the required discharge of the sub-main.

TABLE 15.—Comparative Values of Storm-Water Flow Estimated for Two Types of the Limiting Rain Curve.

For $R = 0.60$.

For impervious areas throughout.

$a = 1$, and $R = 0.60$.

Increase of storage, S_m, in cubic feet per acre.	For a Limiting Rain Curve of the San Francisco Type.		When the Limiting Rain Curve is a Parabola.	
	t_m, minutes.	d_m, second-feet per acre.	t_m, minutes.	d_m, second-feet per acre.
200	3.89	1.43	2.72	1.82
250	6.08	1.17	4.25	1.46
300	8.75	0.99	6.11	1.21
400	15.6	0.77	10.9	0.91
500	24.2	0.64	16.9	0.73
600	35.0	0.54	24.4	0.61
700	47.5	0.47	33.4	0.52
800	62.0	0.42	43.6	0.15
900	78.6	0.38	55.0	0.40
1 000	97.3	0.35	68.1	0.36

Or, as may frequently be convenient, the sub-main is considered separately. In this event it must be remembered that the mass curve of surface run-off, which is of most unfavorable shape for the main district, is as shown in Fig. 8. During the first part of the critical period, the rainfall will have its greatest intensity. The run-off produced by the rain of greatest intensity will have time to escape from the sub-district (supposed to be relatively small) before the main conduit flows at capacity. Therefore, in applying the graphical method, only that part of the run-off curve is to be used which lies nearest to T. But, for decreasing intensities of rain, in a small district, there will probably be less water in the conduits at the time, T, than t_D minutes before T (t_D now applying to the sub-district). Consequently, instead of an augmentation of water quantity stored in the conduits there will be a depletion of this storage. The value of $C_T - C_B$ or of S, as the case may be, will be negative, and must be platted upward from the curve at T, instead of downward.

In the supposed case, Fig. 8, the sub-district time is 20 min. and the required discharge of the sub-main is represented by the inclination of the line, $E_1 D_1$.

It may be of interest to note that the described method of estimating flow at any point of a water conduit when the rates are known at which

water is supplied to it by its feeders, is applicable to rivers and canals* as well as to covered conduits, and also to conduits operating in part under pressure.

A closed conduit serving throughout under pressure is an extreme case. There can then be no augmentation of the quantity of water stored in the conduit. The effect of an increase or decrease in the rate of supply will be instantaneous from end to end of the conduit. The time, t, will become zero. The points, B, T, and E on the curve will all be at the origin of the co-ordinates, and the maximum rate of flow at all points of the conduit will be equal to the maximum rate at which water is supplied, because, for $t = 0$, the E-curve is tangent to the supply curve at the point, B.

At the other extreme is the case of flow through large reservoirs or basins. In this case, the storage of water is large, the time, t, is relatively long, and the maximum rate of outflow is correspondingly small.

In applying the method of estimating required storm-water capacity, as described, it is well to remember that a proper determination of the value of b eliminates the necessity for introducing into the calculation any further allowance for safety margin except such as may be covered by a selection of a value for the factor a. As a is unity for areas with impervious surfaces throughout and as the value of a is otherwise definitely determinable, it will be seen that the room for error is small.

General Deductions.—It appears from the foregoing:

1.—That the method as described of estimating storm-water flow is applicable to districts of any shape, size, or surface condition.

2.—That if a conduit affords no reservoir space, the storm-water flow at all points thereof would be equal to the rate of inflow at the storm-water inlets, and the effect of an increase or decrease of this rate would be immediately manifest at all points of the conduit. (This is the case of a conduit under pressure.)

3.—That the longer the time required for storm-water concentration, the larger will be the unit storage capacity of the conduits in any district, or *vice versa,* and the greater the influence of the conduit storage upon the rate of flow.

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* This is substantially the method of estimating river flow described by the writer in a recent discussion of a paper on a California flood. *Transactions*, Am. Soc. C. E., Vol. LXI, p. 332.

4.—That an outfall conduit, receiving no further accessions, below a certain point:

a.—If occasionally or always submerged must be given uniform capacity from that point to the point of outfall;

b.—If not flowing under pressure, and if it have a free outfall not obstructed by back-water, its maximum discharge will decrease somewhat with increasing distance below that point.

If the shape of the limiting rain curve has been properly determined from adequate rainfall observations, the secondary curves derived therefrom should be smooth curves. The value of discharge determined therefrom will, in other words, follow a definite law of change.

It is obvious that the graphical method of determining discharge may be applied to any rain storm for which a mass curve of the rainfall can be platted. The proper location of the points, *B* and *T*, on the resulting surface run-off curve will be found by trial.

The method of estimating storm-water flow as here described commends itself for use whenever the limiting rain curve is known:

First.—Because the maximum fall of rain in all possible time intervals is taken into account.

Second.—Because the element of uncertainty relating to the rate of rain-water inflow into storm-water inlets is confined within narrow limits, and the possible error in the selection of a reduction factor is correspondingly small.

Third.—Because the essential element, the increase of water storage in the conduit system during a critical period of time, is subject to determination with reasonable accuracy.

Fourth.—Because it is simple and generally applicable.

Storm-Water Relief Outlets and Sewer Capacities.

The ordinary principle of providing storm-water relief outlets from the sewers of the combined system, where opportunity offered has been adhered to in designing the storm-water conduits for San Francisco. The infrequency of rain in excess of ¼ in. per day made it appear desirable to let the relief outlets come into service only when the rainfall rate exceeded this amount. In other words, the sewers were planned to carry the run-off due to a rain of ¼ in. in 24 hours, to the sewerage outfall point. The remainder of the flow during storms will pass through the relief outlets on the shortest course to bay or ocean.

The sewers of the combined system, therefore, were given a capacity above relief outlets to carry:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall of greatest intensity;

Some allowance as a safety margin.

Below relief outlets:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall at the rate of ¼ in. in 24 hours.

The sewer capacity thus determined for points below relief outlets is nearly four times as great as the mean flow of sewage proper. The storm-water reliefs, therefore, should not come into service with more than 25% of sewage in their outflow, and this amount only at times when their outflow is small. When in service at full capacity the sewage carried by them will be so dilute as to be quite inoffensive.

Sewage Disposal.

Dilution and Outfall Points.

Consideration was given to the various methods of sewage disposal which have been developed and found satisfactory.

Utilization by application to land was entirely out of the question, owing to San Francisco's location at the extreme point of a narrow peninsula, and the remoteness of lands which would be suitable for cultivation under sewage irrigation.

Treatment in filter beds, to produce an inoffensive effluent, or chemical treatment for like purpose, would entail much additional cost and is not at present necessary for any part of San Francisco's sewage, owing to the great dilution that can be secured, and the favorable set of the currents from the selected outfall points away from the shore.

Dilution in the waters of the bay and the ocean is without doubt for San Francisco the natural and proper method of sewage disposal.

The questions to be considered, therefore, relate to the best points of outfall and the best method of conveying the sewage to those points.

Without discussing at length the studies of tidal currents made in 1893 with floats, and the results of current measurements made from anchored boats by the United States Coast and Geodetic Survey some years before, the conclusions reached will be briefly stated.

The float observations did not cover the entire water front, nor all possible stages of the tide. The means were not at command to accomplish this. Generally known facts or opinions relating to the direction and velocity of the bay currents led to the selection of certain points from which the floats were started at various stages of the tide and then followed day and night by boat until stranded or well started seaward. The float was of wood about 6 ft. long, weighted at one end and carrying a flag or lantern at the other.

Float observations were made from points off Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. At each of these points floats were started at low tide, at high tide, and at intermediate stages. There were no float observations made oceanward from Golden Gate as these did not seem necessary. The current in and out is swift. Someone has computed that 1 cu. mile of water passes through Golden Gate each way every day. This vast volume, as it flows outward, entrains to some extent the water that lies off the beach, southward from Point Lobos. This is probably the reason why the ocean beach is exceptionally free from drift.

All the Hunter's Point flood-tide floats passed up the bay, clear of tide marsh and shore line. The ebb-tide floats cleared Potrero Point by more than half a mile and passed east of Mission Rock.

Floats started at flood tide from the Sugar Refinery wharf at Potrero Point took a course inside of a line drawn from Potrero Point to Hunter's Point; one was caught and held in the flood-tide eddy south of Potrero Point. At ebb tide all floats passed to the east of Mission Rock into the deep water of the bay.

The floats started at flood tide from the wharf of the Pacific Rolling Mill, farther north on Potrero Point, all hugged the shore toward Hunter's Point. The ebb-tide floats showed equally unfavorable conditions. Their course was close to the end of Center Street pier, to the westward of Mission Rock.

Floats started at flood tides from the end of Center Street pier, oil wharf, passed close to Potrero Point. Those at ebb tide moved toward Mission Rock.

The floats started at the end of Powell Street pier on flood tides in part took a course somewhat off shore into deep water, in part they went gradually shoreward. The ebb-tide floats all went off shore more or less, most of them clearing Black Point by more than one-quarter mile.

The conclusion reached from the foregoing observations was that sewage might safely be discharged into the bay or ocean on the northern frontage of the city, provided only that the outfall be arranged far enough off shore in deep water.

The eastern frontage of the city, from near Market Street to near Hunter's Point, is not a desirable locality for disposing of sewage. It was recognized, however, that the discharge of sewage in limited amounts through submerged outlets in deep tidal waters is rarely objectionable, and, therefore, that small shore areas which cannot at once be made tributary to the contemplated main sewers, may, with proper restrictions, be allowed to discharge into the bay.

Hunter's Point was found to offer exceptional advantages for a delivery of sewage into swift tidal currents with ample and thorough dilution.

The following points of outfall were finally selected:

Off North Point, with discharge in at least 36 ft. of water at 1 200 ft. or more from the water-front line.

Off Hunter's Point, in a corresponding depth.

Off the ocean shore just west of Baker's Beach, at the foot of 27th Avenue.

Off the foot of Scott Street, in at least 36 ft. of water, and well off shore.

Two other points were selected, though not to be utilized at once. These are Fort Point, where it may be found desirable after many years to concentrate all sewage reaching the northern city front, and Point Lobos, where it was foreseen that the sewage of the Sunset and other ocean-slope districts should ultimately be delivered.

Of the areas tributary to the selected outfall points it may be said:

The North Point main is to receive the sewage from an area of about 11 500 acres, which includes the greater portion of the residence district of the city and the entire business section. About 80% of the city's population was within this area at the time the report was written in 1899.

The territory tributary to the Hunter's Point main comprises 2 300 acres.

The Point Lobos outfall will ultimately receive the sewage from the greater portion of the ocean slope of the city.

The Scott Street outfall will serve the Harbor View district, until it becomes desirable to extend the North Point main to Fort Point.

The conclusion having been reached that the best points for the discharge of sewage are the projecting points of the water front which are swept by swift tidal currents, it became necessary to give Fort Point special consideration. Fort Point, at the south side of Golden Gate, is without doubt the best location for an outfall, but, for the present, it is inaccessible owing to remoteness from the built-up section of the city, and the great cost that would be involved in reaching it in advance of general water-front improvement.

In the order of their desirability as points of outfall for the sewage of bay-slope districts there seems to be no question that Fort Point should be ranked first, though, as explained, not now available; North Point, second, and Hunter's Point, third.

Consequently, the collecting system of sewers has been planned so as to make the greatest area possible tributary to the North Point outfall, and let Hunter's Point take care of the remainder.

This is the fundamental reason why, as elsewhere explained, the upper portion of the Islais Creek drainage is to be cut off from the Hunter's Point system and made tributary by gravity flow to the North Point main, by tunneling along San José Avenue under College Hill. By this arrangement, moreover, an immediate installation of the outfall sewers for an important outlying district becomes possible, and it has the further advantage of reducing the amount of sewage to be pumped over from the Hunter's Point main into the North Point sewer system if, in the future, it ever becomes necessary to abandon the Hunter's Point outfall. Furthermore, it appears unobjectionable to permit the temporary delivery of sewage in small amounts at other points of the bay front, provided the discharge be always effected well off shore in deep water.

Sewerage Districts.

Sewerage districts were thereupon laid out in conformity with these principles and the plans for the systems of collecting conduits were completed.

The areas noted in Table 16 and the estimated population, which is really the assumed distribution of population when San Francisco has reached the 1 000 000 mark, were used as guides in estimating, by methods already described, the quantity of sewage, soil-water, and storm-water to be cared for in the sewers.

TABLE 16.—SAN FRANCISCO SEWER DISTRICTS.

Name of district and sub-district.	Area, in acres.	Estimated future population.
North Point District:		
County Line West	836	21 800
County Line East	381	8 500
Persia Avenue	246	7 300
Sunnyside	579	21 700
Glen Park	582	17 500
Silver Avenue West	177	5 300
Valley Street and Upper Army Street	794	31 700
West Bernal Heights	149	6 000
Lower Islais Creek	1 125	47 900
West Potrero, Upper Twenty-second Street	620	31 000
Eighteenth Street	660	33 000
Fourteenth Street	1 307	71 500
Miscellaneous	27	1 600
Eleventh Street	978	54 400
East Potrero	334	16 700
Fifth Street, Sixth Street, Seventh Street and Miscellaneous	782	103 700
Kearny Street	41	6 200
Mission Flats	845	72 500
North Rincon Hill	126	15 700
Portsmouth	157	31 000
Yerba Buena	279	33 500
North Beach	500	30 000
Totals	11 525	674 500
Hunter's Point District:		
College Homestead	122	3 700
Gaven Street	68	2 000
St. Mary's	91	3 600
South Bernal Heights	123	4 900
University Mound	488	14 600
West Silver Terrace and Miscellaneous	65	2 000
North Silver Heights	77	3 100
Railroad Avenue	115	4 600
North Slope, Hunter's Point	306	12 200
Bay View	590	23 600
South Slope, Hunter's Point	249	10 000
Totals	2 294	86 300
Harbor View District	816	48 100
Richmond	1 200	60 000
West Richmond	212	19 200
Upper Sunset	1 658	64 200
Lower Sunset	2 564	98 600
Ocean View	783	29 800

POPULATION, WATER CONSUMPTION AND SEWAGE.

Before describing the district systems, a word is to be said relating to the prospective growth of San Francisco, and the water consumption. The population estimate, as made in 1899, is shown by the diagram, Fig. 11. For comparison, the population of several other cities of the United States is also shown. The conclusion was reached that about 50 years would elapse before San Francisco would have a population of 1 000 000. Fifty years being a reasonable time for which to insure adequate service to all parts of the city; and fully realizing that no

forecast now made for a period so remote is to be regarded other than as a probability, this population was accepted as that on which estimates of water consumption and, therefore, volume of sewage proper and the like, should be heard.

The information presented in Fig. 11, and extended to include the results of the census of 1900, is shown in Table 17.

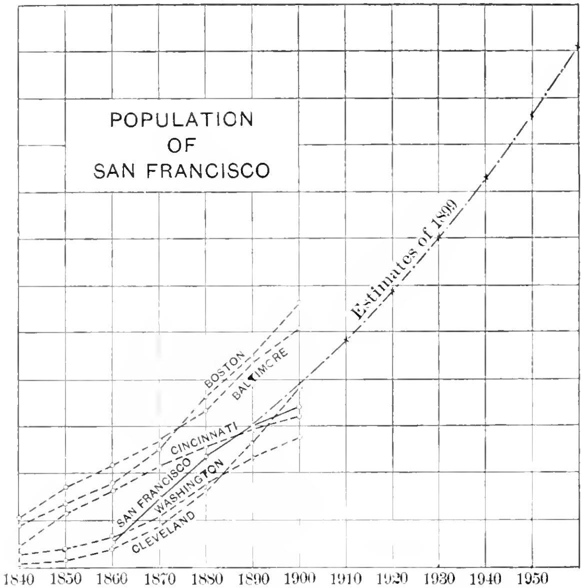


Fig. 11.

TABLE 17.—Population of Cities from United States Census, and Estimated.

Year.	Washington.	Cleveland.	Cincinnati.	San Francisco.	Baltimore.	Boston.
1840	23 364	6 071	46 338	....	102 313	93 383
1850	40 001	17 034	115 435	....	169 054	136 881
1860	61 122	43 417	161 044	56 802	212 418	177 840
1870	109 199	92 829	216 239	149 473	267 354	250 526
1880	177 624	160 146	255 139	233 950	332 313	362 839
1890	230 392	261 383	296 908	298 997	434 439	448 477
1900	278 718	381 768	325 902	342 782	508 957	560 892
1910	....	....	....	*480 000	....	....
1920	....	....	....	*590 000	....	....
1930	....	....	....	*700 000	....	....
1940	......	....	....	*830 000	....	....
1950	....	....	....	*960 000	....	....
1960	....	....	....	*1 100 000	....	....

* Estimates made in 1899.

As a result of the destruction of the business section and a large part of the residence district of San Francisco by fire in 1906, there was at that time a sudden drop in the actual population curve.

If recast at the present time there would, no doubt, be some departure from these estimates.

From information available in 1899, it was found that, exclusive of water pumped from bay or ocean for the supply of bathing tanks, for use in street sprinkling, and for use in condensers, about 70 gal. per capita per day were supplied to the city by the Spring Valley Water Company. The maximum daily per capita consumption at that time was 82 gal.

It is believed that San Francisco's cool climate, particularly in the summer, the compact development of the residence districts, with restricted lawn and garden areas, and the natural difficulties in the way of bringing in a copious supply, will always compel more than ordinary economy in the use of water. It is not likely, therefore, that the present consumption of 75 gal. per day will ever be greatly exceeded. Allowing 20% as the excess of the maximum daily consumption over the mean, and making a small allowance for water taken from ocean, bay and private wells (4% was assumed), the conclusion was reached that 94 gal. per day should be taken into account in estimating the volume of sewage proper.

About 75% of the water used for all purposes in the city will reach the sewers. But the rate at which the water is used and delivered to the sewers is not uniform. It was assumed, without special local studies to confirm the accuracy of this assumption, that the maximum flow would exceed the mean by 50%.

Based on these premises, it was estimated that for each 1 000 inhabitants the average quantity of sewage proper, on days of maximum water consumption, would be 0.109 sec-ft., and that the maximum rate at which this would reach and be carried off by the sewers would be 0.163 sec-ft. This was the unit adopted for use in estimating the required capacity.

Soil-Water.

The quantity of soil-water entering the sewers by leakage depends on the care with which the sewers are constructed, their permeability, leaky joints, and also the relative elevation of the water-table and sewer. When the water-table is below the sewer there may be leakage from the sewer into the ground, but there will be no reverse flow.

Throughout a large portion of San Francisco the sewers will lie above the soil-water plane. If winter rains bring the water-table up to cover the sewer, in the high parts of the city, it will be for short periods only. Therefore, very little inflow from the soil into the sewers is to be assumed, except in the low marginal and down-town areas, where the plane of saturation lies above the sewers. It would be interesting to know whether this plane could not be permanently lowered, in view of the fact that the bay front of the city is, or was, skirted by mud flats which are to be considered nearly impervious. But this cannot be ascertained short of very expensive experiments.

If there is a great leakage into the sewers, they may have some effect on the water-table, and the leakage will gradually decrease until equal to the ground-water replenishment, from whatever source this may come. In the absence of adequate information relating to ground-water depth in different parts of the city, a uniform allowance for an increment of the flow in the sewers due to this cause was made for all the high portions of the city and a larger allowance for the low areas. The ground-water reaching the sewers from high ground was introduced into the calculation of sewer capacity at 0.001 sec-ft. per acre, and that from the low parts of the city, where the sewers, for the most part, will lie below the natural soil-water plane, at 0.003 sec-ft. per acre.

The Tides of San Francisco Bay.

The range of tides in San Francisco Bay is thus referred to in the *Pacific Coast Pilot*, by George Davidson, Hon. M. Am. Soc. C. E., who for many years was in charge of the work of the United States Coast and Geodetic Survey on the Pacific Coast:

"From the lower low water ('low water large') the tide rises for about 7½ hours, say 4.4 ft. to the smaller of the two high tides ('high water small'), then falls 1.4 ft. in less than 4½ hours to the 'low water small,' which is higher than the preceding low water; then rises say 2.9 ft. in 6¼ hours to the higher high water or 'high water large;' it then falls again 5.8 ft. in over 7 hours to the lower low water, or 'low water large.'

"Instead of the above figures, the fall from high water small, or 'half tide' to the 'low water small,' may range from 3½ ft. at one position of the moon to 0.3 ft. at another; in the latter case there will be apparently a long stand of about 5 hours.

* * * * * * * * *

"The average difference of the higher high and lower low waters of the same day is 5.2 ft., with a greatest observed range, noted for February 8th, 1876, of 9.93 ft."

The "Tide Tables" for 1899 of the United States Coast and Geodetic Survey gave the following information:

The expected mean range of tides is 4.5 ft.

The expected mean range of spring tides is 5.4 ft.

The expected mean range of neap tides is 3.5 ft.

The expected mean range of great tropic tides is 7.2 ft.

The expected maximum range of tides in 24 hr. is 7.8 ft.

The expected minimum range of tides in 24 hr. is 3.1 ft.

From these tidal ranges, and the area of the bay, together with the area and elevation of the adjoining submersible land, and with proper allowance for the difference in the time at which the high and low tides occur, the total volume of flow through the Golden Gate can be estimated.

To the outflow through the Golden Gate, however, there is to be added the flow of the rivers tributary to the bay. This is not inconsiderable.

It is the run-off from more than 60 000 sq. miles of tributary area. The two rivers, the Sacramento and the San Joaquin, contribute more than 4 000 sec-ft. at their lowest autumn stage, and more than 80 000 sec-ft. throughout the first six months of the year, often reaching from 150 000 to 200 000 sec-ft. for short periods of time.

River flow through the bay alone would effect sewage dilution from 30 to 1 500 fold, yet the river water is only a trifle when compared with the great return flow of waters which have entered the Golden Gate on a rising tide.

North Point Sewer District.

The North Point Sewer District has an area of 11 525 acres. Its estimated population, when that of the city, of present area, is 1 000 000, will be 674 500.

About 80% of the city's population in 1899 lived within this district. It includes the northern frontage of the city easterly from Black Point, the eastern frontage as far south as the hills on the south side of Islais Creek, and extends inland to the summit of the peninsula ridge and along the same, south to, and slightly beyond, the county line, including all the Islais Creek water-shed already described.

Nearly all this district is to be sewered on the combined system. The old sewers are utilized to the greatest extent possible.

The position of the main intersecting sewer was determined by two governing points, the outfall at North Point and the point on the old line of Mission Creek where the sewage of the Mission is to be intercepted. The sewer was placed low enough to serve the district near Harrison and Fourteenth Streets. At that point many street grades have been established without due regard to future requirements, and to the permanent injury of adjoining property. Improvements of street surface and improvements on private land have been made to conform to these grades, many of which are too low. It is doubtful whether material correction of such grades will ever be possible. The North Point main, therefore, was planned to fit the grades as established, the aim having been to keep the hydraulic grade line at least 1 or 2 ft. below the street surface at the critical points. This was the best that could be done for this district which, under present conditions, is submerged whenever there is a heavy downpour of rain. In order that the reason for small safety margin, for the scant elevation at a few points of street surface above the hydraulic gradient of the full conduit, may be better understood, it may be stated that there were such points, as at Eighteenth and Division Streets, where the grades marked out a pot-hole arrangement. The official grade there at the time the sewer system was planned was 7.5 ft. above city base,* while at all street intersections around this point the official grades ranged from 10.5 to 13 ft. At Thirteenth and Harrison Streets there is a similar arrangement. Street grades of 6 and 6.5 ft. are entirely surrounded by official grades several feet higher. Official street elevation is similarly defective at the northern termination of York, Florida and Alabama Streets, and at the intersection of Fifteenth and Shotwell Streets; also at Sixteenth and Shotwell Streets; at the eastern termination of Fifteenth Street, on Folsom Street midway between Sixteenth and Seventeenth Streets, and at Fourteenth Street; and there are still other points where grades should be changed so as to eliminate completely the pot-hole feature.

In the past this district has suffered one inundation after another. The proposed increased capacities of the sewers and storm-water drains

* The grade at this point was raised a few years after the report was written.

will greatly ameliorate this condition, but the mischief done by the establishment of faulty street grades cannot be entirely undone by the construction of sewers, and it must be left to the property owners to find out how much better off they would be if the high-water line in the sewers could be kept 5 or 6 ft. below the street surface instead of rising almost to it. The only satisfactory relief from the local floods that may result from defective or choked storm-water inlets and from the annoyance of water in cellars, or, at least, of interrupted cellar drainage during severe rain storms, will result from raising the street grades, particularly at the points where pot-hole features are to be eliminated. The present property owners are often the innocent sufferers, from the desire of their predecessors to have official grades established at natural ground height or but little above it, in order to save the cost of grading.

The head of the North Point main, if considered as an interceptor, is far out on the Mission Road, at the point, in fact, where this road crosses Islais Creek. From this point the sewer will be in tunnel for a distance of 4 500 ft. following the line of Mission Road and San José Avenue.

At Army and Valencia Streets the sewage now flowing down Army Street sewer is to be intercepted and diverted northward. The present sewer eastward from this point will continue in service as a storm-water relief outlet, and as a main from which a mile farther east sewage will be pumped to the nearest point of the North Point main.

The course of the North Point main is almost due north following the streets best adapted by reason of elevation to receive it. It will be on Twenty-sixth Street from Valencia Street to Treat Avenue, on Alabama Street from Twenty-fifth to Nineteenth Streets, and will reach Division Street at Eighteenth Street. It will follow Division Street to Harrison Street, paralleling the present Channel Street sewer; thence it will be on Harrison Street to Eighth, on Eighth to Howard, thence on Howard and New Montgomery or Second Street northwest to Market Street and north to the water front on either Montgomery or Sansome Street. All along its course it will intercept the sewers entering it from the west.

At Eighteenth and Division Streets, at Fifteenth Street, and about on the line of Fourteenth Street, will be storm-water relief outlets of large capacity. The outlet arrangement at Eighteenth Street is practi-

cally that of two parallel conduits with a common roof, separated from each other by a dividing overfall weir. The entire flow will be confined by this weir to the North Point main until the full capacity is reached, whereupon the excess flow will go over the weir into the storm-water conduit.

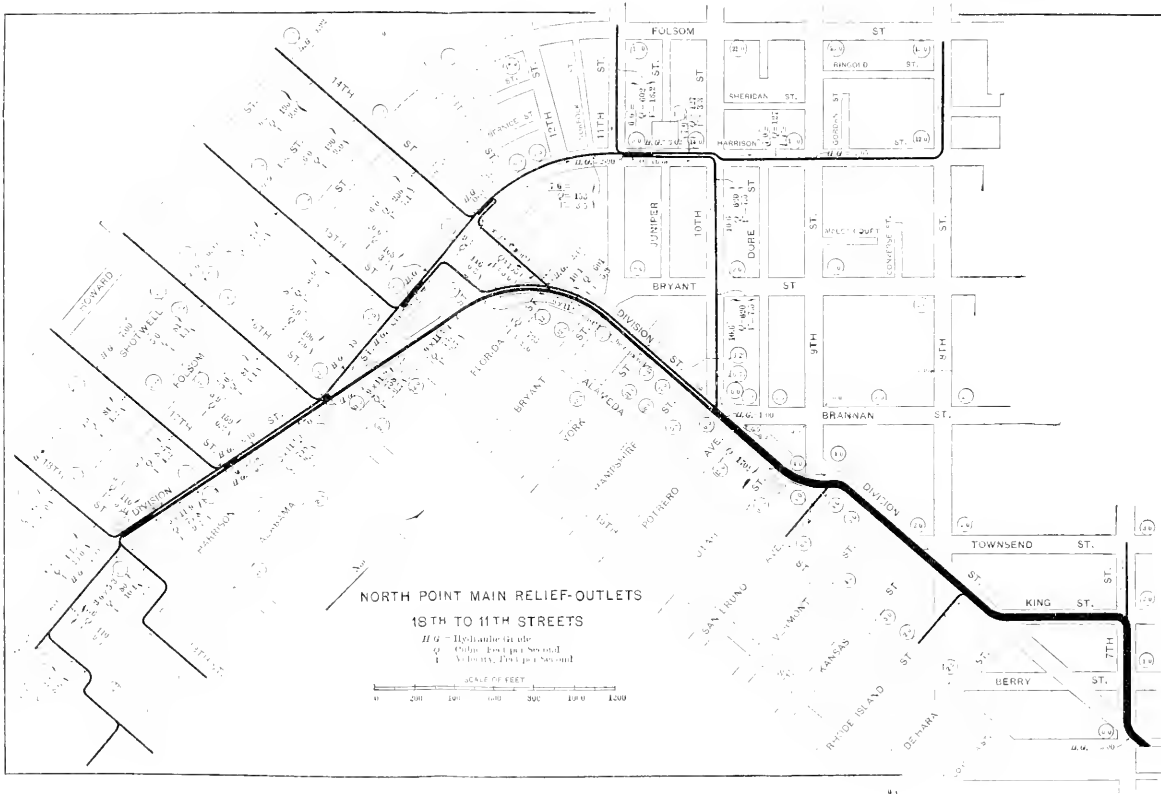
Other storm-water outlets will be provided at Seventeenth, Sixteenth, Eleventh, Seventh, Sixth, and Fifth Streets, also at Mission Street and at Commercial and Jackson Streets, as well as a final one at the North Point screen-house, where the sewage will enter the outfall pipes.

Large conduits will carry the water from each of the storm-water outlets in the most direct line to the open water in Channel Street, or to the bay. Of these conduits, the only one of special interest is the Division Street conduit which will have its head at Eighteenth and Division Streets, but will be the recipient also of the storm-water overflow at Seventeenth, Sixteenth, Fifteenth, and Fourteenth Streets, and at several other points along its course to the head of the open water in Channel Street.

This conduit, shown on Plate IX, will be called upon to carry ultimately about 760 sec-ft. near Eighteenth Street, and about 1 700 sec-ft. at its outfall end. It will be a conduit 9 ft. high and 11 ft. 9 in. wide from Eighteenth Street to Tenth Street, being paralleled by a second conduit from Alameda Street to Tenth Street. Thence to its outfall, for a distance of about 2 000 ft., the conduit will be constructed with two or three compartments. Three, each 9 ft. wide, will be required ultimately. It may be found advisable, however, to construct only two of these at the outset. Owing to the fact that the floor of the conduit must be kept as high as possible on account of the unfavorable character of the ground along its route, and because the street surface along the line of this conduit is lower than desirable, scant headroom is available, and the conduit has been planned with a flat top of reinforced concrete.

It has been suggested by interested citizens that the main storm-water outfall should be at the foot of Sixteenth Street, and by some it is even contended that the sewage of the entire area that can be made tributary to the intersection of Sixteenth and Division Streets should be carried in a large conduit along Sixteenth Street to the bay.

Taking Fourteenth and Harrison Streets as the initial point for



H G = Hydraulic Grade
Q = Cubic Feet per Second
V = Velocity, Feet per Second

such an outfall, it can readily be seen by a glance at any map of the city that the distance thence to Sixteenth Street and along Sixteenth Street to the end of Center Street wharf would be twice as great as the distance from the same point along Division Street to the head of the open water in Channel Street. Consequently, a conduit along Sixteenth Street would have less gradient than a storm-water outlet along Division Street. It would have to be of larger cross-section, and the velocities in it would be less than if located along Division Street. In other words, for conduits of similar dimensions, on the two routes, there would be more probability of surface inundation at and near Fourteenth and Harrison Streets if the conduit were located along Sixteenth Street than if located as planned.

The foot of Sixteenth Street does not meet the requirements of a permanent outfall for the sewage of the area that can be made tributary to it. Nothing that can be delivered at North Point should be allowed to reach the water at such a central point on the eastern bay frontage.

Attention may be called to the fact that, in line with this requirement, the sewage of the eastern portion of the Potrero Hills is to be collected in a secondary interceptor which will skirt the northern base of these hills and connect with the North Point main at Ninth and Howard Streets. The storm-water overflow from this secondary East Potrero interceptor will flow in suitable conduits to the Division Street relief outlet.

The North Point main will be in tunnel under Telegraph Hill for 2 500 ft., if it follows Montgomery Street. If located on Sansome Street, the tunnel construction, if required at all, would be much shorter.

In its largest section, the sewer was planned to be circular, 8 ft. in diameter, with a concrete invert and a brick arch. In doubtful ground it is to be supported on piles. Double sheet-piling on each side is to facilitate trenching. The piles used as guide piles for the sheeting are to be left in place as a part of the sewer foundation, and the sheet-piling on either side of the trench is to serve as a part of the form for the concrete invert. Longitudinal steel rods are to be placed in the concrete, and as many piles as may seem necessary for support are to be driven in the trench and capped with the concrete of the sewer invert.

The hydraulic gradient prescribed for the sewer when flowing

full will have an elevation of 7.00 ft. above City Base at Eighteenth and Division Streets, dropping to 3.50 ft. below City Base at the outfall. Its gradient is about 0.0005, or 1 in 2 000. The sewer invert, as originally planned, was to have gradients gradually changing from 1 in 1 800 near Division and Eighteenth Streets to 1 in 2 000 at the outfall.

The hydraulic grade line at the North Point screen-house will be 0.50 ft. above extreme high water of the bay and 3.20 ft. above ordinary high water. The sewer invert at the same point will be about 12.90 ft. below City Base, or a little more than 6 ft. below ordinary high tide.

During the rising tide, therefore, the discharge of sewage will be retarded; during the falling tide it will be accelerated. The effect of this will be a reduced outflow into the flood tides and an increased discharge into the ebb, which is a desirable feature.

The area of the city on the bay front which lies to the eastward of the North Point main will be treated in three subdivisions, from each of which sewage will be pumped into the North Point main.

It is not proposed in this paper to enter upon a minute description of the system of sewers outlined for these and for the many other main and sub-districts, because the problems are in large part the same as are presented in all growing cities. Only those features of the problem will be referred to which appear somewhat unusual or may otherwise be considered of special interest.

Yerba Buena and Mission Flats Sub-District.—The Yerba Buena sub-district, which extends from Telegraph Hill on the north to Rincon Hill on the south, and lies between the North Point main and the bay, is a low, flat, for the most part filled-in, section of the city. It is throughout an important business district.

The storm-waters of this sub-district are to be delivered to the bay for the most part in the sewers now in use, but remodeled as far as practicable to give them continuous slope toward their outfall points.

Sewage proper will be provided for in separate conduits leading to a pumping station between Commercial and Sacramento Streets, a short distance east of Drumm Street. The pumps will send the sewage westward into the North Point main.

The Mission Flats sub-district, 845 acres in extent, which embraces the old Mission Bay region as far as it lies below or east of the

North Point main, will also be sewered on the separate system. The pumping station for this district is to be located north of Channel Street, near Fifth Street. The sewage south of the Channel will be carried under the open waterway in an inverted siphon. The pumps will send the sewage northwestward along Fifth Street into the North Point main.

In both these districts the sewers of the separate system will be ironstone pipes. The smallest lateral sewers are to be 8 in. in diameter and the minimum gradients on which these are to be laid, with but few exceptions, will be 1 in 200.

The principal objection to the use of the combined system in these districts is the fact that the surface of the districts lies low. Very much of it is at City Base. The efficient drainage of basements has not been possible with the combined system heretofore in use. Better service was demanded. It was necessary to choose between, on the one hand, a combined system of sewers leading to pumping stations, with storm-water relief outlets to the bay and intermittent pumping, and, on the other hand, the complete separation of the sewers from the storm-water conduits. In the former case there would have been times, during rain storms, when the sewers would have been full to a plane well above high tide in the bay, and back-flow into basements unguarded by check valves would have been inevitable. From this situation there was no remedy except either to provide pumping capacity on a combined system great enough to handle the maximum storm-water flow, or to introduce the separate system. The latter plan was adopted, but it, too, has its disadvantages. One is the duplication of conduits in streets already crowded with pipes for gas and water, and with conduits for telephone and telegraph wires and wires for transmitting electricity. It was found necessary in many places to provide sewers for each side of the street.

The main drawback to this system is probably to be found in the fact that some of the streets of the filled-in areas are still in a condition of subsidence. On this point there was very little information available in 1899. It was known that the lower end of Market Street, which had been paved many years before at official grade, was then several feet low, and the various buildings with their sidewalks, which had been constructed at grade, had settled measurably. Nothing was known concerning the continuance of the subsidence. The general

opinion seemed to prevail that but little more was to be expected. The adopted system, under these circumstances, included provision for the support of the sewers on piles. It was proposed to place the sewers in a continuous, pile-supported bed of reinforced concrete wherever the ground was doubtful.

In the light of later information, it now seems uncertain whether any type of construction can be devised that will hold the sewers at grade.

As City Engineer during 1900 to 1904 the writer made studies of street subsidence. Careful levels were taken at intervals of one year. The results of these studies for the two years, 1901 to 1903, are shown in brief form in Table 18.

TABLE 18.—SUBSIDENCE OF STREETS IN SAN FRANCISCO.

Location.	Subsidence, in Feet.	
	1901–1902.	1902–1903.
Davis Street, Market to East Streets	0.05	....
Jackson Street, Montgomery to East Streets	0.03	....
Spear Street, Market to Bryan Streets	0.05	....
Mission Street, First to East Streets	0.05	0.02
Harrison Street, Fourth to Seventh Streets	0.15	0.19
Sixth Street, Howard to Brannan Streets	0.05	0.05
Sixth Street, Brannan to Channel Streets		0.08

From April, 1902, to April, 1903, there was no appreciable subsidence on Davis Street except a small amount at its intersection with Vallejo Street, where the subsidence during the preceding 12 months had been 0.08 ft.

From April, 1902, to April, 1903, there was no appreciable subsidence on Jackson Street except a small amount at Drumm and East Streets, where the subsidence during the preceding year had been 0.06 ft.

From April, 1902, to April, 1903, there was subsidence on Spear Street only at Mission Street, where the subsidence during the preceding 12 months had been 0.06 ft.

The subsidence is not confined to the streets alone, but includes adjacent buildings; even those within the area under consideration that are supported on piles 90 ft. and more in length show measurable subsidence.

The lower end of Market Street in 1903 was from 2 to 3 ft. below

official grade from curb to curb, the street-car tracks supported on piles having settled with the remainder of the street.

As subsidence of the character here referred to includes every structure in or adjacent to the street, no provision for the support of sewers would be adequate to prevent settling. If the sewers are laid to grade there will be sections dipping below grade after a few years, and the dip will increase until after a time the departure from the true gradient may be so great that reconstruction of some of the down-town sections will be necessary.

The earthquake of 1906 has occurred since the subsidence studies were commenced. No information is at hand at this writing as to the effect which the violent shaking has had upon the general elevation of the street surfaces in the made-land districts. Local upheavals of a foot or more, and local depressions of the ground's surface have been noted in various parts of the city. Where these occurred they were more or less destructive to all conduits in the streets, and would have caused serious derangement of the sewer system as planned if it had been in service. It is probable that whatever changes have been produced in the street levels, by the earthquake, have brought them nearer to a permanent condition of stability. At any rate, the probability of the recurrence of earthquakes of sufficient force to prove destructive to sewers is too remote to be taken seriously into account in planning the system. The location of the areas where a derangement is most likely to occur, however, has been pretty clearly indicated by the breaks in conduits of all kinds in the earthquake of 1906, and this knowledge may here and there lead to some modification of the location of a sewer or to some modification of design.

Lower Islais Creek District.—A dual treatment has been planned for the Lower Islais Creek district. From this district sewage proper is to be pumped through a long force main into the North Point sewer. Where there are no sewers now in use the collecting system will be separate from the storm-water drainage; but where sewers have already been constructed to serve for sewage and storm-water combined these are to remain in service. The arrangement at the pumping station, which will be centrally located, will be such that excess storm-water will pass through a relief outlet to Islais Creek. Whenever the excess flow is long continued, which will be coincident with a full North Point sewer, the pumping from the sewers on the combined

system will be stopped. The sewers of the separate system are to reach this pumping station at 15 ft. below City Base. This low grade is made necessary by the large tributary area with low-surface elevation. At a mean stage of water in the receiver at the pump the total lift, including friction in the discharge main, will be about 50 ft.

Only those sewers of this, as of other, districts that lie within improved territory or that are at once required to reach pumping stations or outfall points were recommended for immediate construction. There are some cases in which the sewers recommended for immediate construction lie along streets crossing the marsh and not now graded. In these cases it was assumed that such streets would be ordered graded as a matter of urgency, and that within a few years they would be ready to receive the sewers.

The North Point Outfall.—The screen-house at the outfall of the North Point main is to be a neat structure of brick or concrete. There will be two settling basins of small diameter, to intercept coarse sand and the broken rock reaching the sewers occasionally with the storm-water from the macadamized streets of the outlying regions. These basins are to be arranged so that one or both may be in service. The removal of material from them will be by dredging, possibly by the hydraulic method.

Alongside of the main sewer, just before it enters the screen-house, there is to be a long overfall weir, with crest at the hydraulic grade line, or a little below this, over which any excess flow of storm-water will drop into a low, flat conduit of brick or concrete leading directly to the bay.

The screen-house will be equipped with screens for the interception of rags, cork, and other floating material that should be removed from the sewage before it is discharged into the bay waters. The outfall pipes are to be connected with the settling chambers, and are to be carried at least 400 ft. beyond the head of a solid pier projected for this point. They were planned to be of cast iron, 5 ft. in diameter. It may be found desirable to substitute reinforced concrete. They are to be placed in a trench excavated by dredging to about 50 ft. below low tide.

Hunter's Point Temporary Outfall.

In the case of Hunter's Point main sewer it is not practicable, owing to the unimproved condition of the streets and their location,

across marsh surface and points of hills, to carry the sewer at once to its final point of outfall. The improvement of the streets along which the sewer is to be built is too far in the future. Their grading for sewers is out of the question on account of cost. The main, therefore, will terminate for a time on the north shore of Hunter's Point, and a discharge will be effected through a submerged pipe well off shore.

Harbor View Sewer District.

The time may come when the demand for keeping the waters of the bay absolutely free from sewage will be so great that, whether offensive to the senses or not, the delivery of sewage on the eastern and northern bay frontage of the city must be stopped. Should this time ever come, the sewers as now planned will remain in service, but pumping will be necessary to send all sewage along the water front to an outfall at Fort Point. The arrangement can there be made to discharge practically all sewage of the bay slope of the city into the ebb tide. In this event the sewage from so remote a region even as Bay View would be delivered by pumping into the North Point main, and would be rehandled by the main pumps located at or near the site of the proposed screen-house. Until this time comes, there will probably always be more or less sewage sent into the bay at points less desirable than those selected as main outfall points.

Until actual experience shows it to be undesirable to send limited amounts of sewage into the bay with delivery into deep water, at a few selected points, the great expense of concentrating all at Fort Point need not be incurred.

It was planned, therefore, to deliver the sewage of the Harbor View District, lying on the northern front of the city, to the westward of Black Point into the bay at the foot of Scott Street. The sewers of this district, as at present, will carry both sewage proper and storm-water. Storm-water relief outlets are planned with a view to bringing existing sewers into use at their full capacity. The drainage system of the district was designed with special reference to the present improvements. The intercepting sewers follow streets along which immediate construction is practicable. When the reclamation of the waterfront property has here been completed, a second line of intercepting sewers of small capacity will be needed. The storm-water relief outlets will serve as local sewers below the upper intercepting line until they

reach the waterfront, where their ordinary flow will follow the lower interceptor and be carried by the same to the main outfall point.

This system of sewerage will have the disadvantage, for the low area close by the waterfront (as yet but partially improved), of affording only imperfect basement and cellar drainage. But the property owners will know the limitations, and can plan their improvements to meet the conditions.

The discharge into the bay will be through a large pipe, about 800 ft. long, terminating in water 40 ft. deep. The capacity of this pipe will be adequate for the ordinary sewage flow, with the usual included small amount of rain-water. All excess flow, as already stated, will go to the bay through storm-water relief outlets.

Richmond Sewer District.

For the Richmond District a sewer project was adopted some years previous to the studies of 1899. The sewage and rain-water are carried in common conduits to an outfall just west of Baker's Beach. At the time this outfall was inspected no trace of sewage at the shore nor of offensive odor could be detected. The strong tidal currents sweeping this part of the shore warrant its use as a point of disposal. The later designs provide for an extension of the outfall farther off shore into deeper water and a completion of such mains as have not yet been constructed.

Other Ocean-Slope Sewer Districts.

The topography of the ocean slope of the city westerly and southerly from Richmond District made a subdivision into four additional districts appear desirable. These are the West Richmond, Upper Sunset, Lower Sunset, and Ocean View Districts. The last named includes the Lake Merced region, and extends southward to the County line.

The problem of handling the sewage of these districts, which for the most part will in time be densely populated, is complicated by the fact that the ocean beach, extending for several miles southward from the Cliff House near Point Lobos, is a pleasure ground of the people, and must be preserved undefiled.

This consideration led the writer and his associate engineers to recommend the separate system of sewerage for all these districts,

and a delivery of all sewage proper into the ocean, off Point Lobos, where it will be so diluted as to disappear completely. From this point, moreover, the oceanward flow is away from and not toward the beach.

Storm-waters were to be collected in separate conduits with outfalls into the ocean at selected points. The principal outfall points tentatively suggested for storm-waters were at the foot of J Street and at the foot of X Street.

It is understood that the plan of treatment here outlined for the Ocean Beach drainages is not to be adhered to. It is now proposed to construct at once an interceptor for both sewage and storm-water which will discharge into the ocean at the Point Lobos outfall point already referred to. Sewage will be liberated in deep water well off shore in the vicinity of Mile Rock.

The rapid improvement that is taking place on the ocean slope has made it appear undesirable to install the works for the temporary delivery of sewage from a West Richmond pumping station into the Richmond mains.

It will be possible to place the proposed main sewer of the ocean-slope districts along V Street to Forty-fifth Avenue, to T Street, to Forty-sixth Avenue, to R Street, to Forty-seventh Avenue, to J Street, to Forty-eighth Avenue, and along Forty-eighth Avenue into the tunnel under Fort Miley Heights.

A low-level sewer will follow the Great Highway northward from X Street to a junction with the main interceptor. Velocities in the low-level sewer are estimated at 2.8 ft. per sec.

All the ocean-slope districts, except portions of Richmond, were but sparsely settled in 1899, but a rate of growth was anticipated that made it imperative to provide means at an early date for a disposal of the sewage from the improved areas. At that time there were improved areas of small extent at and near the beach, both to the north and to the south of the Park, and Upper Sunset was fast becoming popular as a residence district. At Ocean View, too, in the southwestern part of the city, a portion of the built-up area was over the divide on the ocean slope and needed attention.

The ultimate point of outfall taken into consideration for this ocean-slope section of the city was Point Lobos, near Mile Rock. The small service required for a number of years, and the high initial

cost of tunneling under Point Lobos Heights, made it undesirable to attempt to reach this point with an outfall sewer at the outset. For this reason the following treatment of the ocean-slope region was recommended. This has since been modified, as already explained.

Upper Sunset District.—The region lying south of Golden Gate Park, between the Park and the Twin Peaks and Blue Mountain group of hills, as far west as Twenty-ninth Avenue, has been designated Upper Sunset District to distinguish it from that part of Sunset which lies at lower elevation and nearer the ocean.

The sewage of this district, under the plan of 1899, was to be carried northward across Golden Gate Park and delivered into the mains of the Richmond District, with outfall at the foot of Twenty-seventh Avenue, just west of Baker's Beach. The storm-waters were to be carried westward in separate conduits.

At the present time both sewage and storm-water are delivered into the Park, as a temporary arrangement, and are being used for irrigation.

Lower Sunset District.—The main western slope of the city, from Golden Gate Park southward, to near Lake Merced, is known as Lower Sunset District. In 1899 the greater portion of this region was covered by high, irregular sand dunes. Streets had long ago been laid out, but official grades remained to be established. At that time, therefore, it was premature to commit the city to a definite scheme of sewer alignment. Since then, however, this region has been one of rapid development, and no mistake was made in 1899 in recommending the immediate construction of works for sewage disposal for which there is growing need. In this district, as for Upper Sunset District, a complete separation of sewage from storm-water was recommended. This plan has now been modified, as already stated.

West Richmond District.—The region known as West Richmond District is practically a part of the Lower Sunset District, separated from the latter by Golden Gate Park. The sewage of both this and Lower Sunset District was to have been led into the receivers of a pumping station located near the ocean on the north line of the Park. It was to be pumped from there temporarily into the mains of the Richmond District, ultimately through a tunnel under Point Lobos Heights into the ocean near Mile Rock. Under the modified plan, the pumping station is rendered unnecessary.

Ocean View District.—Ocean View District is the area sloping westward from the peninsular ridge, in the southwestern part of the city, toward Lake Merced. It embraces parts of Ocean View, Lake View and Ingleside. It is a sparsely populated district, but needs outfall sewers at once because its drainage is toward Lake Merced, which is still in use as a source of water for the city and will probably be used indefinitely, as an emergency source.

The storm-waters of this district will necessarily follow natural drainage lines westward to points where they can be intercepted for diversion past Lake Merced. Some temporary works for such diversion, with permanent tunnel outfall, have already been constructed by the Spring Valley Water Company.

Sewage proper will be collected in separate conduits, and will be carried in a closed main to a discharge into the main sewers of the Lower Sunset District.

Pumping Stations.

The details for the pumping stations were worked out by Mr. H. C. Behr, Mechanical Engineer. Centrifugal pumps were prescribed for all pumping stations, although, for use at the West Richmond station, vertical, crank-driven, triple-plunger pumps were taken into account, to be used until the low delivery through the tunnel at Point Lobos could be effected. Under the modified plans there will be no West Richmond pumping station.

The receiver capacities were fixed, in a measure, to fit the available ground area. If necessary, they may be added to in the future. The receivers were planned circular in outline, in triplicate for each station, and it was assumed that all would have to be carried below the water-table, and that the pneumatic method of construction would be necessary. The receivers at each pumping station are to be interconnected and arranged for use separately or in conjunction. The sewage, before entering the receivers, is to be screened. Ventilation is to be provided by a shaft or chimney, in which a small fan blower, with a multiple injector nozzle, is to create a draft.

All pumps are to be belt-driven. Small units are suggested for the pumps in order to secure the greatest possible adaptability to all requirements. The motors were only tentatively selected for cost-estimate purposes, it being apparent that gas, oil, and electric motors are

all well suited for the work required. Steam engines require more space and a greater cost of attendance. They are objectionable on account of the smoke nuisance, and are not as well suited to the intermittent character of the work that may be required.

The force or discharge main from each pumping station is to be of cast iron. Additional facts relating to the pumping stations are given in Table 19.

TABLE 19.—SEWAGE PUMPING STATIONS.

Name of station.	Required capacity, in second feet.	Total lift, including friction, in feet.	Ultimate horse-power required.	Present horse-power.	Capacity of receiver, in gallons.	Length of force main, in feet.	Diameter of force main, in inches.
Yerba Buena.......	6	31	42	35	60 000	2 100	16
Mission Flats.......	15	35	120	80	60 000	4 000	18
Lower Islais Creek.	12	65	180	88	60 000	4 700	18
Bay View..........	5	30	35	14	40 000	1 650	8

TYPES OF SEWERS.

Concerning the types of sewers to be used, but little need be said. Ironstone pipes are to come into use up to diameters of 2 ft. Sewers of the next larger size, and up to 3 ft. 6 in. by 5 ft. 3 in., are to be egg-shaped. Larger sewers are to be circular, except as stated in the case of the main relief outlet to Channel Street, which is to be flat—the lower end in two or possibly three compartments, as already explained, each nearly rectangular, 9 ft. wide and $8\frac{1}{2}$ ft. high.

The egg-shaped sewers and the large circular sewers were planned to have concrete inverts with reinforcement to give added strength and stability. They were designed without vitrified brick invert lining.

It is understood that the plans have recently been modified somewhat in the matter of sewer types. The use of reinforced concrete is to be extended. All sewers of greater diameter than 2 ft. are to be constructed of this material. Vitrified brick is to be used for invert lining of some of the sewers.

It is to be stated in this connection that recent examinations by the City Engineer of concrete sewers in Sixth Street and H Street that were constructed about five years ago show that these sewers, except for a slight wear in the invert of the H Street sewer, are in excellent condition. No deleterious effect of sewage upon the concrete could be noted.

Whether the tendency, so generally apparent throughout the country, to build the sewers with concrete shells of minimum thickness, which is sure to be attended with many failures, is to be followed or not, remains to be seen.

The Bond Issue.

The report of 1899 was accompanied by a cost estimate covering the construction of the sewers and storm-water conduits immediately required. The expenditure of \$4 600 000 was recommended.

At an election, held soon after the adoption of the report, bonds in this amount were voted to carry forward the work. Before these bonds were issued there was a modification of the form of government in San Francisco. Theretofore the city was subject to control by the State Legislature. Legislative enactment, in the early history of San Francisco, had consolidated City and County. Amendments to the Consolidation Act were thereafter the convenient means of securing privileges in the city of laying out new streets, and of prescribing and limiting the powers of the Board of Supervisors and the other City and County officials.

The charter which was adopted in November, 1899, modified all this, and, among other things, created a Board of Public Works, with usual duties; but this charter, which went into effect on January 8th, 1900, made no provision for carrying to completion the proceedings relating to bond issues commenced under the Consolidation Act. The matter was carried into the courts, whose decision was to the effect that the sewer bonds already voted could not be issued. New proceedings were necessary.

During the first few years under the new charter such matters as the bonding of the city for improvements moved slowly. Each step had to be subjected to the test of the courts. It was not until 1903, therefore, that a new order was passed by the Supervisors directing the City Engineer again to prepare plans for a comprehensive system of sewers.

In accordance with this requirement, the plans of 1899, with but slight modification, were re-submitted by the writer, then City Engineer, under date of June 30th, 1903. The cost estimate was increased because it covered sewers not recommended for immediate construction 4 years before, and because there had been considerable advance in the price of materials, and particularly in wages. The estimated cost of the sewers and accessory structures, then recommended for construction, was \$7 250 000.

The question relating to the issuance of sewer bonds in this amount, together with bonds for other purposes, in the aggregate about \$17 000 000, was submitted to the voters in November, 1903. The bonds for sewers were again voted. But the bonds as authorized were to bear only $3\frac{1}{2}\%$ interest. This low rate of interest coupled with the fact that the government of the city had meanwhile passed into the hands of officials who did not command public confidence, made the sale of these bonds impossible. At length further attempts to sell bonds of this issue were abandoned, and the City Engineer was called upon for a new report. This was after the great calamity of 1906. Restriction of the sewer work to the main lines of sewers was demanded, in order to keep the bond issue as low as possible. The latest recommendation, therefore, does not cover all the sub-mains that were included in the recommendations of 1903, nor even the full system of 1899.

The final submission, in May, 1908, to the voters, of the proposition to expend \$4 000 000 on the sewer system again resulted favorably by a vote of about 15 to 1. Under the bond issue thus authorized the systematic construction of the projected main sewers and interceptors has now been commenced.

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PAPERS AND DISCUSSIONS

THE DESIGN OF ELEVATED TANKS AND STAND-PIPES.

By C. W. Birch-Nord, Assoc. M. Am. Soc. C. E.

To be Presented May 5th, 1909.

Introduction.

It is the writer's intention to submit herewith a general specification for the design of elevated tanks and stand-pipes, which he wishes adopted by the Engineering Profession. As far as he knows this subject has never before been treated for this purpose, and it is therefore his most sincere wish that experienced engineers in this field will offer their most severe criticisms, in order to make the final revised specification a sound guide. Attention may be called to the fact that this specification has been used by the writer for some time, and has always been applied successfully.

The writer wishes to express his obligations to A. F. Reichmann, M. Am. Soc. C. E., and Mr. J. H. Hoff, for assistance in the preparation of this paper.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

General Specifications for Stand-Pipes, Tanks and Towers.

Loads.

1.—The dead load shall consist of the weight of structural and ornamental steelwork, platforms, roof construction, piping, etc.

2.—The live load considered shall be the contents of tanks, the movable load on platforms and roofs, and the wind pressure.

3.—The weight of water shall be assumed to be 63 lb. per cu. ft., and that of crude oil 56 lb. per cu. ft., 1 cu. ft. of fluid being equal to 7.48 gal.

4.—The live loads on platforms and roofs shall be taken at 30 lb. per sq. ft., or a 200-lb. concentrated load applied at any point.

5.—The wind pressure shall be assumed at 30 lb. per sq. ft., acting in any direction. The surfaces of cylindrical tanks exposed to the wind shall be calculated at two-thirds of the diameter multiplied by the height.

6.—The movable live load on platforms and roofs shall not be considered as acting together with the wind pressure.

Unit Strains.

7.—All parts of the structure shall be proportioned so that the sum of the dead and live loads shall not cause the strains to exceed those given in Table 1.

TABLE 1.

	Pounds per square inch.	
Tension in tank plates	12 000	Net area.
Tension in other parts of structure	16 000	" "
Compression	16 000	Reduced.
Shear on rivets and pins	12 000	
Shear on bolts and field rivets	9 000	
Shear in plates	10 000	Gross section.
Bearing pressure on rivets and pins	24 000	
Bearing pressure on field rivets	18 000	
Fiber strain in pins	24 000	

8.—For compression members, the permissible unit strain of 16 000 lb. shall be reduced by the formula:

$$p = 16\,000 - 70 \frac{l}{r}$$

where p = permissible working strain in compression, in pounds per square inch;

l = length of member, from center to center of connections, in inches;

r = least radius of gyration of section, in inches;

the ratio $\frac{l}{r}$ shall never exceed 120 for main members and 180 for struts.

9.—Stresses due to wind may be neglected if they are less than 25% of the combined dead and live loads.

10.—Unit strains in bracing and other members taking wind stresses may be increased to 20 000 lb. per sq. in., except as shown in Section 9.

11.—The pressures given in Table 2 will be permissible on foundations and bearing plates.

TABLE 2.

	Tons per square foot.
Soft clay	1
Ordinary clay	2
Dry sand and dry clay	3
Hard clay	4
Gravel and coarse sand	6
	Pounds per square inch.
Brickwork with cement mortar	200
Portland cement concrete	350
First-class sandstone	400
First-class limestone	500
First-class granite	600

Details of Construction.

12.—The plates forming the sides of cylindrical tanks shall be of different diameters, and shall be caulked from the inside. No foreign material shall be allowed when caulking.

In oil-tank work, both the inside and the outside of the tank shall be beveled for caulking.

13.—Joints for horizontal seams and for radial seams in the spherical bottoms of tanks shall preferably be lap joints.

14.—For vertical seams lap joints shall be used for $\frac{1}{4}$, $\frac{5}{16}$, and $\frac{3}{8}$-in. plates; double butt joints for $\frac{7}{16}$, $\frac{1}{2}$, $\frac{9}{16}$, $\frac{3}{4}$, and $\frac{13}{16}$-in. plates; and triple butt joints for $\frac{7}{8}$, $\frac{15}{16}$, and 1-in. plates.

15.—Rivets, $\frac{5}{8}$ in. in diameter, shall be used for $\frac{1}{4}$ and $\frac{5}{16}$-in. plates; rivets, $\frac{3}{4}$ in. in diameter for $\frac{3}{8}$ to $\frac{5}{8}$-in. plates, inclusive; and rivets, $\frac{7}{8}$ in. in diameter, for $\frac{11}{16}$ to 1-in. plates, inclusive.

16.—Plates more than $\frac{5}{8}$ in. thick shall be sub-punched and reamed.

17.—The minimum thickness of the plates for the cylindrical part shall be $\frac{1}{4}$ in. The thickness of the plates in spherical bottoms

shall never be less than that of the lower ring in the cylindrical part of the tank.

18.—The facilities at the plant where the material is to be fabricated will be investigated before the material is ordered.

19.—All plates shall be punched before being bevel-sheared for caulking.

20.—Radial sections of spherical bottoms shall be made in duplicates of the number of columns supporting the tank, and shall be reinforced at the lower parts, where holes are made for piping.

21.—When the center of the spherical bottom is above the point of connection with the cylindrical part of the tank, there shall be provided a girder at said point of connection to take the horizontal thrust. The horizontal girder may be made in connection with the balcony. This also applies where the tank is supported by inclined columns.

22.—The balcony around the tanks shall be 3 ft. wide, with a $\frac{1}{4}$-in. floor-plate, and shall have a suitable railing, 3 ft. 6 in. high.

23.—The upper parts of spherical bottom plates shall always be connected on the inside of the cylindrical section of the tank.

24.—In order to avoid eccentric loading on the tower columns, and local stresses in spherical bottoms, the connections between the columns and the sides of the tank shall be made in such a manner that the center of gravity of the column section intersects the center of connection between the spherical bottom and the sides of the tank. Enough rivets shall be provided above this intersection to transmit the total column load.

25.—If the tanks are supported on columns riveted directly to the sides, additional material must be provided in the tank plates riveted directly to the columns to take the shear. The shear may be taken by providing thicker tank plates or by reinforcement plates at the column connections, while bending moments shall be taken by upper and lower flange angles. Connections to columns shall be made in such a manner that the efficiency of the tank plates is not less than that of the vertical seams.

26.—For high towers, columns shall have a batter of 1 to 12. The height of the tower is understood to be the distance from the top of the masonry to the connection of the spherical bottom, or the flat bottom, with the cylindrical part of the tank.

27.—The bottom plates of stand-pipes shall be not less than $\frac{5}{16}$ in.

thick, and shall be provided with tapped holes, $1\frac{1}{4}$ in. in diameter, with screw-plugs spaced at 4-ft. centers to allow a filling of cement on top of the masonry, while the bottom part is being erected, in order to secure the proper bearing.

28.—Near the bottom of the stand-pipe there shall be provided one 12 by 18-in. manhole of elliptical shape.

29.—Near the top of each tank and stand-pipe there shall be provided one Z-bar acting as a support for the painters' trolley and for the stiffening of the tank. The section modulus of the same shall be not less than $\frac{D^2}{250}$, where D is equal to the diameter of the tank, in feet. If the upper part of the tank is held by the roof construction, this may be reduced.

30.—On large tanks, circular stiffening angles shall be provided in order to prevent the tank plates from buckling during windstorms. The distance between the angles shall be located by the following formula:

$$d = \sqrt{t}\ \frac{900}{D}$$

where, d = approximate distance between angles, in feet;
t = thickness of tank plates, in inches;
D = diameter of tank, in feet.

31.—The top of the tank will generally be covered with a conical roof of thin plates; and the pitch shall be 1 to 6. For tanks up to 22 ft. in diameter, the roof plates will be assumed to be self-supporting. If the diameter of the tank exceeds 22 ft., angle rafters shall be used to support the roof plates.

Plates of the following thicknesses will be assumed as self-supporting for various diameters:

$\frac{3}{32}$-in. plate, up to a diameter of 18 ft. 0 in.
$\frac{1}{8}$-in plate, up to a diameter of 20 ft. 0 in.
$\frac{3}{16}$-in. plate, up to a diameter of 22 ft. 0 in.

Rivets in the roof plates shall be from $\frac{1}{4}$ to $\frac{5}{16}$ in. in diameter, and shall be driven cold. These rivets need not be headed with a button set.

32.—A trap-door, 2 ft. square, shall be provided in the roof plate. Near the top of the higher tanks, a platform with a railing shall be provided, for the safety of the men operating the trap-door.

33.—An ornamental finial shall be provided at the top of the roof.

34.—A ladder, 1 ft. 3 in. wide, shall be provided from a point about 8 ft. above the foundation to the top of the tank, and also one on the inside of the tank. Each ladder shall be made of two $2\frac{1}{2}$ by $\frac{3}{8}$-in. bars with $\frac{3}{4}$-in. rungs. On large, high tanks, 30 ft. or more in diameter, a walk shall be provided from the column nearest the ladder to the expansion joint on the inlet pipe.

35.—In designing tanks, 6 in. additional height shall be allowed for over-run.

36.—The bracing in the towers shall be adjustable.

37.—The size of the anchor-bolts shall be determined by the uplift when the tank or stand-pipe is empty. The unit strains in the anchor-bolts shall not exceed 15 000 lb. per sq. in., and the minimum section shall be limited to a diameter of $1\frac{1}{4}$ in.

38.—The concrete shall be assumed to have a weight of 140 lb. per cu. ft., and shall be sufficient in quantity to take the uplift.

39.—Any parts of the tank, stand-pipe, or tower, in which difficulties may arise in field riveting, shall be assembled in the shop, and marked properly before shipment.

40.—The structural material shall conform to the "General Specifications for Steel Railroad Bridges" by the American Railway Engineering and Maintenance of Way Association.

41.—The workmanship shall be in accordance with the Manufacturers' Standard Specifications of February 6th, 1903.

42.—Before leaving the shop all work shall be painted with one coat of approved paint, excepting the laps in contact on the tankwork. All parts which will be inaccessible after erection shall be well painted. After erection, the structure shall be covered with one coat of the same paint.

43.—Three-ply frost-proof casing shall be provided, if necessary, around the inlet pipe. This casing shall be composed of two layers of 1 by $2\frac{1}{2}$-in. lumber, and each layer shall be covered with tar paper, and one outside layer of $\frac{7}{8}$ by $2\frac{1}{2}$-in. dressed and matched flooring. The lumber shall be in lengths of about 12 ft. A 1-in. air space shall be provided between the layers of lumber, and wooden rings or separators shall be nailed to them every 3 ft. The frost casing may be made square or cylindrical.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A SYSTEM OF COST KEEPING.

By Myron S. Falk, Assoc. M. Am. Soc. C. E.

To be Presented May 5th, 1909.

The description of itemized costs of contract work always attracts some attention; unfortunately, the greatest amount of publicity has been given to the actual costs of unit items on construction work rather than to the bookkeeping methods used in obtaining those quantities.

It seems hardly necessary to call attention to the very small value that cost records may have to others than to those who gathered them and who realized and understood fully every local condition at the point of construction. There are some, however, who believe that all cost records have value, and that their publication, no matter in what form, will be a benefit. There is no doubt that the records of cost of completed work are useful to some slight extent to others than their compilers, but the greatest value of such records is derived during the time of their collection by the men who are doing the work.

By their use, a contractor can determine with reasonable accuracy the value of the work performed during any specified period; he can compare its cost with that of any other period, and he can determine

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

whether the efficiency of his force is increasing or decreasing and so take the necessary steps to keep his organization in good order. It must be common experience that the superintendent at the point of construction is always an optimist; that he gauges his output for any week by the best day's work, and estimates unit costs by that gauge; his memory is short, with regard to delays and bad weather, and his estimate of the growth of work in the field is prone to error.

Cost records, therefore, have a distinct and important value to their compilers, and this paper, for which no special originality or singular merits are claimed as to the system of bookkeeping which it describes, is presented that others may describe their methods, with a view of developing some simple and uniform systems of accounting, so that a contractor may know for what purpose he is making expenditures.

The Accounting System.—The company keeps but one book, namely, a cash book; in this book there are entered on opposite pages the receipts and payments of money. The distribution of the payments to their various accounts is the basis of cost keeping.

Every payment, no matter of what kind or for what purpose, is made in the form of a blanket voucher, as shown by Figs. 1 and 2; each payment as made is numbered consecutively in the cash book, and this number appears on the voucher and on every bill, order, or blank which concerns in any way the payment in question; the tracing of orders, material received, etc., is checked by noting whether a voucher number is marked upon it.

On the back of the voucher (in the proper blank space), after the bill has beeen receipted, there is written the account or item against which the payment is to be charged. This payment may be charged against any item of any particular contract, or against any general or overhead expense account, such as traveling, rent, or telephoning. The charge on the back of this voucher is entered in the cost-keeping ledger, which is a loose-leaf book, a sample page of which is shown by Fig. 3. Each page represents one item, and particular attention is called to the left half of the page, which represents expenditures. These expenditures are itemized in two groups and a "Total"; the first column, headed "Labor," represents labor charges; the second column shows any and every other expense chargeable to the main item; and the third, headed "Total," shows the sum of the preceding two, whenever desired. It is evident that this page is the key to the

New York 190

Co.,

To .... Dr.

As per bill attached.

PLEASE DATE AND SIGN THIS AND RETURN PROMPTLY,
TOGETHER WITH ATTACHED BILLS, ALSO RECEIPTED.

Correct as to calculation,

....

Checked and approved,

....

Received 190

.... Dollars

in payment of above account.

....

....

$ ....

PLEASE FOLD THIS VOUCHER BUT ONCE.

PLEASE RETURN THIS VOUCHER RECEIPTED IMMEDIATELY.

Voucher No. ....

.... Co.

Name ....

Date 190

$ ....

CHARGEABLE TO

General Expense

Salary

Sundry

Traveling

Other Equipment

Contract at

Plant

Engineering

Labor

Supplies and Materials

Sundry

Total

Signature examined and voucher filed by } ....

PLEASE FOLD THIS VOUCHER BUT ONCE, AND ON THE LINE.

Standard Size, 7 by 8½ in.

FIGS. 1 AND 2.

Sheet No. 3

________ CONSTRUCTION CO. CONTRACT ________

CHARGE TO ITEM NO. 3 Eight-Axed Granite Masonry

DATE		VOUCHER NO.	EXPENDITURES	LABOR		TOTAL	DATE	RECEIPTS	TOTAL
Mar	11		Unloading Lighter Pier III	17 50					
"	12		Loading for Pier II Course 4	17 50					
"	13		Rain all day	6 45					
"	14		Laid Pier III A 387 cu. ft.	36 52					
"	15		" " B.C. 237 " "	28 65					
"	19		Rain and snow	7 20					
"	20	624	F. Baldwin Manila rope		31 40				
"	"	630	Red Star Towing Co. towing		30 —				
"	"	631	O. J. Stephens — Coal		131 25				
			etc						

Standard Size 9 in. wide and 12 in. high.

FIG. 3.

Foreman..........................

Timekeeper..........................

Date..........................

NAME	HOURS	RATE	NAME	HOURS	RATE

ENGINEER'S DAILY REPORT

To be forwarded to main office every evening

.... COMPANY,

DATE ....
WEATHER ....
TEMPERATURE ....

LOCATION OF WORK .... SECTION NO. .... CLASS OF WORK ....

Work Commenced .... A.M. Work Ceased .... M Work Recommenced .... M Work Ceased .... M.

Class of Labor	Foremen	Laborers I	Laborers II	Laborers III	Laborers IV	Masons	" Helpers	Bricklayers	" Helpers	Carpenters	Timbermen	Dock Builders	Steam Engine Men	Firemen	Teams	Carts									Total
No. of Men																									
Total Hours Worked																									
Pay per Hour																									
Amount																									

ROSENDALE / PORTLAND } CEMENT USED .... BAGS

COAL RECEIVED

Material Received this Day. (Forward receipt tickets with this report.)

SIGN HERE

....ENGINEER

Standard Size, 4 by 6 in.

FIGS. 4 AND 5.

entire system; the cost of labor is maintained entirely distinct from every other charge, and that cost is precisely the one most desired, as other costs, for plant, raw material, or anything else, can, as a rule, be determined or estimated without essential error.

Before describing the entries in this book, it is necessary to state that as soon as the company receives a contract, it prepares a list showing the items under which all expenses are to be subdivided. If the contract is based on unit prices, these items include the various units of the contract; for example, piles, or grillage timber, or waterproofing, etc., as well as items for plant erection, or plant removal, or plant maintenance, which cover expenses that cannot be charged against any one unit until the completion of the work. Again, many of the unit items of the contract are in turn subdivided.

CONTRACT OF____________ CO., AT________ | TEMPERATURE MAX'M.____ MIN'M.____ | TIME ____ | DATE ______ 190 | SHEET NO.

CLASSIFICATION	NO. OF MEN	TOTAL HOURS	RATE OF WAGES	CHARGE TO	CHARGE TO	CHARGE TO	CHARGE TO	CHARGE TO	CHARGE TO	CHARGE TO	CHARGE TO	TOTAL
TOTAL												

Standard Size, 5 by 8 in.

FIG. 6.

As soon as a blanket voucher is receipted and returned to the company, it is charged against the proper item in the cost-keeping ledger, and is then filed, the only exception being in the case of vouchers for pay-rolls. As one pay-roll voucher covers either one or two weeks' work, it obviously cannot be entered in the ledger until it is subdivided into unit charges. This is performed in the field by the timekeeper, who uses the cards shown in Figs. 4, 5 and 6. Fig. 4 shows the pay-roll card for each day; each man's name or number, together with the amount earned by him, is entered upon this, and, on the reverse side (Fig. 5) is tabulated all the labor for the day. This tabulation is then carried forward to the card shown by Fig. 6, and is there itemized and charged against the various items of the contract. If the pay-roll is met weekly, seven of these cards

CONTRACT OF CO., AT

TEMPERATURE MAX'M MIN'M | TIME | DATE 8/29/08 190 | SHEET NO.

CLASSIFICATION	NO. OF MEN	TOTAL HOURS	RATE OF WAGES	CHARGE TO Gravel Side Walk	CHARGE TO 2" Pipe and Railing	CHARGE TO Asphalt Blocks	CHARGE TO Plant Const.	CHARGE TO Maintenance	CHARGE TO #2 Concrete	CHARGE TO Curbing	CHARGE TO Ashler Masonry	TOTAL
Foreman	1	8				(11)	5 00				1 (2)	5 00
Laborers	15	1 hrs	20	5 60	16 80	33 10	(32)		25 60			81 10
" 2	4	28¾		(34)	(42)	3 34	2 06			(32)	1 27	6 67
" 3	2	16	35			2 80		(45c.)	2 80			5 60
Masons	2	12		1 75	1 75				(12)		2 75	6 25
D. & B.	8	64	43¾				14 00	14 00				28 00
St. Engineers	2	16				4 20	3 75					7 75
St. Cutters	2	8	56¼							4 50		4 50
Curb Setters	2	8	"							3 75		3 75
Boy & Horse	1	8				3 60						3 60
Oscars "	1	8				2 00						2 00
Watchman	1	–		25	25	25	25	25	25	25	25	2 00
Timekeeper	1	–			25	25	25	25	25	25		1 50
Super	1	–		75	1 00	50	50	1 00	1 00	50	25 OK	5 00
TOTAL	79			7 85	20 05	50 04	25 81	15 50	29 90	9 25	4 52	8 162.92

Laid 30 sq. yds. Telford

Laid 380' of 2" pipe and set 6 railing posts South approach East Side

Unloaded 23,950 asphalt Blocks

Fitting up derrick Pier 4

Moved Stock House

Mixed 61 1/3 sq. yds. of road foundation

Set 691 – Cut 15–16"

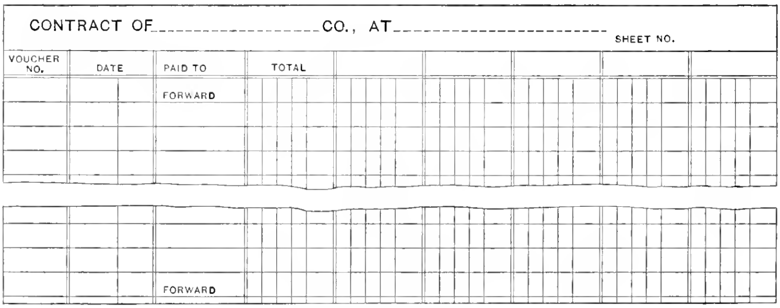
Cleaning and pointing Pier 6

Standard Size, 5 by 8 in.

FIG. 7.

must equal the total shown on the blanket voucher of the cash book. There is also written on the itemized pay-roll cost card (Fig. 6) the amount of work accomplished on each item; the quantities are obtained by the timekeeper each day from the various foremen without any difficulty and with comparatively small error. As an example, Fig. 7 is reproduced; this is an actual card turned in by the timekeeper. As soon as the pay-roll and blanket voucher are made up, the items on the cards representing that roll are entered in the cost-keeping ledger, as shown by Fig. 3.

The items in this book are totaled in pencil, at intervals of a few days or a week, in order to note the progress and cost of the work; each page is permanently totaled and checked in ink on the first day of each month, and checked against a supplementary card-system, which



CONTRACT OF__________________CO., AT______________________ SHEET NO.

VOUCHER NO.	DATE	PAID TO	TOTAL				
		FORWARD					
		FORWARD					

Standard Size, 5 by 8 in.

FIG. 8.

records the total cost of any contract, in the condensed form shown by Fig. 8. On this card, each blanket voucher is also entered as soon as filled out, and is itemized into one of five general headings; usually these headings are (1) Labor, (2) Plant, (3) Material Remaining in Work, (4) Other Material, and (5) Sundries, there being five columns on each page. This supplementary card ledger is extremely useful in showing at any time the general divisions of expense.

To complete the description of the system, the card illustrated by Fig. 9 is shown. This card is the summary of seven of the daily cards shown by Fig. 4, and is used in making up the weekly pay-rolls.

It has been found that the cards shown by Figs. 4 and 6 might be combined in one card in order to save the time of the timekeeper, but the sheet resulting from the combination has been found

to be too bulky to be used on a stiff card, in which form all the records of the company are now kept, and, rather than use thin paper records, the present card system is maintained.

.... COMPANY. Time for Week Ending ....

Location of Work .... Section No. .... Sheet No. ....

Class of Work ....

Names / Date																								
S.																								
M.																								
T.																								
W.																								
T.																								
F.																								
S.																								
No. of Hrs.																								
Rate																								TOTAL
Amount																								

Note:- On back of this sheet, furnish description of work accomplished this week. SIGN HERE ....

Standard Size, 4 by 6 in.

FIG. 9.

As used by the company, the foregoing system has furnished the office force precisely the information which it needed at any time in order to trace loss of efficiency in the field; and the records of the completed work, particularly in respect to the item of labor, are in the form needed when other and similar work is estimated. The system described is that used by the Godwin Construction Co., of New York.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

NICKEL-STEEL EYE-BARS FOR BLACKWELL'S ISLAND BRIDGE.*

By William R. Webster, M. Am. Soc. C. E.

Having obtained some very interesting results, in connection with the preliminary experimental tests of carbon and nickel-steel built-up riveted eye-bars and forged nickel-steel eye-bars, and in the tests of the eye-bars used in the Blackwell's Island Bridge, it seemed to the writer that it would be desirable to give a summary of this work, and to place on file in the Library of the Society, a report, with tests, etc., where it would be accessible to those interested.

The eye-bars used in the structure were the largest ever manufactured, some of them being 16 by $2\frac{1}{8}$ in., with heads $37\frac{1}{2}$ in. in diameter, with 16-in. pins, and more than 64 ft. from center to center of pin holes. Special machinery had to be made for upsetting and rolling the eye-bar heads, and a testing machine of 2 000 tons capacity had to be designed and built to break them.

Nickel steel had been used for years in forgings, with special heat treatment. The best results were obtained by oil-tempering and annealing. When it was proposed to use nickel steel for eye-bars, it

*Additional data on this subject are filed in the Library of the Society, where they may be examined by any one who is interested.

This paper will not be presented at any meeting, but written communications on the subject are invited for publication with it in *Transactions*.

was apparent that the hot bars would be distorted in handling during the process of oil-tempering and annealing, and it became necessary to learn what results could be obtained by modifying the ordinary methods of annealing carbon-steel eye-bars.

The first nickel-steel eye-bars were manufactured by The American Bridge Company at Edge Moor, and were tested at Pencoyd, in 1902. There were twenty-one of these bars, 6, 8 and 10-in., of basic open-hearth steel containing 3.26% of nickel. Records of the tests of these bars are filed in the Library of the Society, and a summary is given in Table 1. The elastic limits were determined by the halt in the mercury column, in the usual way.

In February, 1903, the Commissioner of Bridges, of the City of New York, requested the writer to make a series of experimental tests of nickel-steel eye-bars, and prepare specifications for steel of different qualities required for the construction of the Blackwell's Island and Manhattan Bridges. Eight bars of basic open-hearth steel, containing 3.24% of nickel, were made and tested at Pencoyd. Records of these tests, with the specifications prepared at that time, are filed in the Library of the Society, and a summary is given in Table 2. The elastic limits of this lot of eye-bars were taken by scribe marks of a finely pointed beam compass on a base length of 10 ft. A summary of the specification referred to above, and all other specifications referred to in this paper, are given in Table 3.

When submitting these specifications the writer stated:

"In preparing these specifications all the tests that had been made at Pencoyd, and other information at hand, were considered. The requirements may seem a little severe, but if proper care is taken in the manufacture of the steel, can all be met.

"In order to insure greater care in the manufacture and uniformity in the steel, high and low limits are specified for ultimate strength in all cases. The leeway of 15 000 lb. seems to be wide enough, and should not have to be increased. If the upper limit is omitted, the same amount of care will not be exercised in rolling the steel, very little attention will be paid to the finishing temperature, higher carbons will probably be used, and reliance will be placed on the results of the annealed test pieces to accept the steel.

"Tension and bending tests are called for on the bars as rolled without annealing. These are additional checks, to insure proper care being taken in the manufacture, and prevent brittle steel being used. If this is not done, and annealed test pieces are relied on alone to

TABLE 1.—Comparison of Specimen Tests of Nickel-Steel Bars, Unannealed and Annealed, with Eye-Bar Tests.

Bars manufactured by Carnegie Steel Co.
Eye-bars manufactured and tested by American Bridge Co. } All from Heat No. 10 450
Analysis: C. 0.21, P. 0.007, S. 0.016, Mn. 0.53, Si. 0.011, Ni. 3.265.

Nickel Steel.		Specimen Tests.				Full-Sized Eye-Bar Tests.				Fracture.
		Unannealed.		Annealed.						
Mark of Bar.	Size of Bar.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Percentage of elongation in 10 ft.	Percentage of reduction.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A 1 *x*	6 by 1 in.	47 815	79 510	49 120	77 140	46 230	74 450	13.2	42.40	Silky, Angular.
A 2 *x*	"	48 190	80 300	50 895	75 890	46 300	74 100	16.2	49.80	" ½ Cup.
A 3 *x*	"	47 330	79 960	47 127	73 320	43 100	69 000	17.7	49.60	" " "
A 4 *x*	"	50 655	79 300	44 460	72 760	45 510	70 420	16.2	54.80	" " "
B 1 *x*	6 by 2 in.	46 963	76 930	48 945	76 920	47 300	72 200	16.3	42.90	" Cup.
B 2 *x*	"	46 615	78 880	48 105	78 260	46 260	74 290	17.4	46.90	" "
B 3 *x*	"	48 220	75 840	47 365	70 060	44 040	68 200	15.3	53.10	" ½ Cup.
B 4 *x*	"	50 335	78 420	48 515	74 170	43 120	69 340	12.7	41.23	" Angular.
B 5 *x*	"	48 215	76 360	51 663	78 790	44 860	72 400	12.1	41.10	" "
C 1 *x*	8 by 1½ in.	......	......	45 805	78 190	42 500	73 600	17.1	41.20	" "
C 2 *x*	"	47 813	76 970	47 730	70 240	40 420	66 150	20.8	57.50	" ½ Cup.
C 3 *x*	"	48 410	79 260	48 620	78 480	43 510	74 720	14.5	51.46	" Cup.
C 4 *x*	"	46 630	77 710	48 005	74 620	48 220	77 080	18.3	48.01	" "
C 5 *x*	"	47 460	79 170	48 650	74 090	41 450	74 450	16.8	47.44	" ½ Cup.
C 6 *x*	"	48 590	79 730	47 146	73 740	43 730	73 620	17.1	49.69	" Angular.
E 1 *x*	10 by 1¼ in	47 840	79 110	48 740	79 240	46 320	72 920	13.7	47.00	" Cup.
E 2 *x*	"	48 650	79 660	48 220	73 440	41 630	69 810	15.3	38.40	" Angular.
E 3 *x*	"	47 130	74 270	48 170	78 110	46 480	76 540	15.5	42.20	" Cup.
E 4 *x*	"	50 020	78 930	47 210	77 380	46 070	74 540	13.1	43.70	" "
E 5 *x*	"	48 320	80 320	47 970	75 710	42 960	73 290	17.1	51.70	" "
E 6 *x*	"	47 620	79 630	48 110	78 070	44 530	75 410	15.1	40.00	" Angular.

Elastic limit of specimen tests taken by drop of beam; of full-sized eye-bar tests by halt in gauge. The figures in Columns 5 and 6 are the results of specimen tests of annealed bars.

TABLE 2.—COMPARISON OF SPECIMEN TESTS OF NICKEL-STEEL BARS, UNANNEALED AND ANNEALED, WITH EYE-BAR TESTS.

Bars manufactured by Carnegie Steel Co. }
Eye-bars manufactured and tested at Pencoyd. } All from the same heat. Analysis: C. 0.35, S. 0.022, Phos. 0.02, Mn. 0.68, Ni. 3.24. Philadelphia, April 4th, 1903.

NICKEL STEEL.		SPECIMEN TESTS.				FULL-SIZED EYE-BAR TESTS.				FRACTURE.
		Unannealed.		Annealed.						
Mark of Bar.	Size of bar.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Elastic limit.	Ultimate strength.	Percentage of Elongation in 18 ft.	Percentage of Reduction.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
H 1 *x*	6 by 2 in.	52 800	90 650	47 040	79 610	47 820	85 000	9.17	42.47	Silky, ¼ Cup. 5% Crystalline.
"	"	51 020	88 370	49 320	76 900					
H 2 *x*	6 by 2 in.	50 740	101 000	48 190	88 600	46 350	83 780	10.76	52.17	Silky, ½ Cup.
"	"	45 190	98 600	47 620	86 050					
H 3 *x*	6 by 2 in.	30 300	98 610	49 090	86 600	48 580	87 190	9.77	38.80	40% Crystalline, Irregular.
"	"	51 250	102 600	47 990	91 450					
H 4 *x*	6 by 2 in.	45 690	106 290	48 670	86 600	48 230	86 130	9.50	41.90	¾ Cup, 25% Crystalline.
"	"	49 780	101 920	48 360	88 860					
H 5 *x*	6 by 2 in.	49 960	102 460	51 760	90 260	48 380	86 490	9.50	44.20	Irregular, 30% Crystalline.
"	"	51 280	99 400	50 520	86 750					
J 1 *x*	6 by 1½ in.	49 950	95 150	46 240	83 140	49 350	86 240	10.44	45.75	Cup, 15% Crystalline.
"	"	50 520	103 420	44 800	86 780					
J 2 *x*	6 by 1½ in.	49 080	96 400	47 110	80 000	49 300	85 490	10.27	44.00	½ Cup, Crystalline, Specks on both edges.
"	"	49 450	99 670	43 180	98 800					
J 3 *x*	6 by 1½ in.	49 190	91 780	44 150	83 810	49 360	86 160	8.44	42.74	¾ Cup, 10% Crystalline on one edge.
"	"	53 390	111 120	44 780	83 600					

Elastic limits taken with dividers; 8-in. gauge length specimens; 10-ft. eye-bars.
The figures in Columns 5 and 6 are the results of specimen tests of annealed bars.

TABLE 3.—SUMMARY OF SPECIFICA-

Specifications.	Unannealed Specimens. Ultimate strength.	Elastic limit.	Percentage of elongation in 8 in.	Percentage of reduction.	Bends.	Annealed Ultimate strength.	Elastic limit.
William R. Webster's original specifications	90 000 to 105 000	55 000	18	35	180° 37'	85 000 min.	52 000
Blackwell's Island Bridge original printed specifications	80 000 to 105 000	52 000	do.	do.	do.	85 000 to 100 000	50 000
Blackwell's Island Bridge contract specifications	100 000 min.	55 000	1 600 000 / ultimate.	to be recorded.	do.	85 000 min.	48 000
Modifications: * Lot or item tests	95 000 to 110 000	do.	do.	do.	do.	do.	do.
Test each heat with high tensile strength	113 000 min.	do.	do.	do.	do.	do.	do.
For bars which break in head	95 000 min.	do.	do.	do.	do.	do.	do.
Conclusions Requirements that can be met as proven by tests of bars for structure.	95 000 to 110 000	do.	do.	35	do.	90 000 min.	52 000

* Tensile in full-sized tests reduced from 85 000 lb. down to 80 000 lb., all other requirements to remain as specified.

Where only one amount is given it means minimum.

check the quality of the material, there is nothing to insure the finished eye-bars being annealed in the same manner and similarly restored. Whereas, starting with a known good material, the annealing will further improve it."

These specifications for nickel-steel eye-bars were submitted to the Board of Engineers, consisting of Messrs. Theodore Cooper, Henry W. Hodge, Mansfield Merriman, George S. Morison, and C. C. Schneider, Members, Am. Soc. C. E., appointed to consider an eye-bar cable for the Manhattan Bridge.

They reported that a nickel-steel forged eye-bar cable, made to these specifications, would be satisfactory to them, except that they considered the elastic limit of 50 000 lb. per sq. in. a little too high,

TIONS FOR NICKEL-STEEL EYE-BARS.

Specimens.			Full-Sized Eye-Bars.					Chemistry.			
Percentage of elongation in 8 in.	Percentage of reduction.	Bends.	Ultimate strength.	Elastic limit.	Percentage of elongation in 18 ft.	Percentage of reduction.	Location of fracture.	Phos. Acid.	Phos. Basic.	S.	Ni.
20	40	180° 2*T*.	85 000	50 000	9	40	In body.	0.06	0.04	0.05	3.25 to 3.50
20	40	do.	do.	48 000	do.	40	do.	do.	do.	do.	3.50
1 600 000 / ultimate.	to be recorded.	do.	do.	do.	do.	to be recorded.	do.	do.	do.	do.	do.
do.	do.	do.	85 000 and 80 000	do.	do.	do.	do.	do.	do.	do.	do.
do.	do.	do.	85 000	do.	do.	do.	do.	do.	do.	do.	do.
do.	do.	do.	do.	51 000	6	do.	In head.	do.	do.	do.	do.
1 800 000 / ultimate.	40	do.	do.	50 000	9	35	In body.	do.	do.	do.	do.

All bending tests are full thickness, and not less than 4 in. wide.

Elastic limits of full-sized tests to be taken by extensometer on base length of 10 ft., a permanent set of 0.025 in. to be considered the elastic limit.

and suggested modifying it to 48 000 lb.; also, that any visible set in the eye-bar should be taken as the elastic limit, using the best means to determine what constituted a visible set—a magnifying glass if necessary.

These specifications were used when the city first asked for bids on the Blackwell's Island Bridge, but the contract was not awarded at that time.

One of the manufacturers questioned the narrow leeway of 15 000 lb., on account of the influence of segregation on the ultimate strength in as large an ingot as would be required to roll the heavy bars. In order to determine what differences would be found, he rolled two bars, from two heats of nickel steel, 18 by 2 in. and 18 by 2¼ in., each more than 65 ft. long. Tests were taken from each end of these

bars to represent the top and bottom of the ingots, with the following results:

18 by 2-in. plate	test cut from top.	96 940 lb.	tensile.
	" " " bottom,	89 000 "	"
18 by $2\frac{1}{4}$-in. plate	" " " top,	82 040 "	"
	" " " bottom,	80 750 "	"

These results were so satisfactory that he agreed that the leeway of 15 000 lb. was sufficient.*

When the city asked for bids the second time, the specifications were changed materially, and a minimum limit of 100 000 lb. tensile, with no maximum limit, was specified.

After the contract for the bridge had been awarded, it was suggested to use built-up riveted eye-bars in place of the forged eye-bars called for in the contract. The writer, having been appointed Inspecting Engineer, was instructed to make a large series of tests, on bars of this kind, of both high-carbon steel and nickel steel, in order to determine their relative merits.

There were tested twenty-two high-carbon steel built-up riveted eye-bars, of the following dimensions:

8	Eye-bars,	24 by	1 in.,	with	8-in. pins.
4	"	24 "	$1\frac{1}{8}$ "	"	8 " "
2	"	24 "	$\frac{3}{4}$ "	"	8 " "
2	"	31 "	$\frac{3}{4}$ "	"	12 " "
6	"	32 "	$\frac{3}{4}$ "	"	12 " "

and twenty-five nickel-steel built-up riveted eye-bars of the following dimensions:

8	Eye-bars,	24 by	1 in.,	with	8-in. pins.
10	"	24 "	$\frac{3}{4}$ "	"	8 " "
7	"	32 "	$\frac{3}{4}$ "	"	12 " "

All the riveted bars had one pin-plate on each side, and the rivets were countersunk. All holes were drilled from the solid, and all sheared edges planed; most of the bars had two lines of $\frac{15}{16}$-in. drilled holes along the body. The center-to-center distances of pin holes were 23 ft. 6 in. and 25 ft.†

From six to eight tension and bending tests were made on specimens from each plate, of both high-carbon and nickel steel, used in these

*A copy of all tests is filed in the Library of the Society.

†Fully detailed drawings of these bars have been filed in the Library of the Society.

built-up eye-bars. Special care was taken in the heating and rolling of these steels.*

In order to determine the elastic limits as accurately as possible, the writer had a special extensometer made. It multiplied ten times, could be relied on to about 0.005 in., and was easily attached to the bar. Fig. 1 shows this instrument and the method of using it.

None of the riveted eye-bars gave satisfactory results, as shown by the detailed reports of the tests.† The loss of strength in the carbon-steel bars, from the average of the specimen tests, was from 11 to 41% on the net area, and from 20 to 46% on the gross area; and, in the nickel-steel bars, from 7 to 13% on the net area, and from 13 to 22% on the gross area.‡

This loss of strength was greater than would be expected from the results of tension tests on small riveted joints or on small narrow bars with drilled holes in them, and is no doubt due to the wide thin plate tearing when tested in tension. Particular attention is called to this, as it has often been suggested to make very large eye-bars from wide plates by machining them narrower in the center in order to prevent the loss of strength which occurs in forged eye-bars due to annealing. Before large bars of this kind are used, a complete series of tests should be made, in order to show what the actual loss in strength would be in a wide, thin plate, as compared with a forged eye-bar of half its width and double its thickness made from steel showing the same tensile strength in the specimen tests.

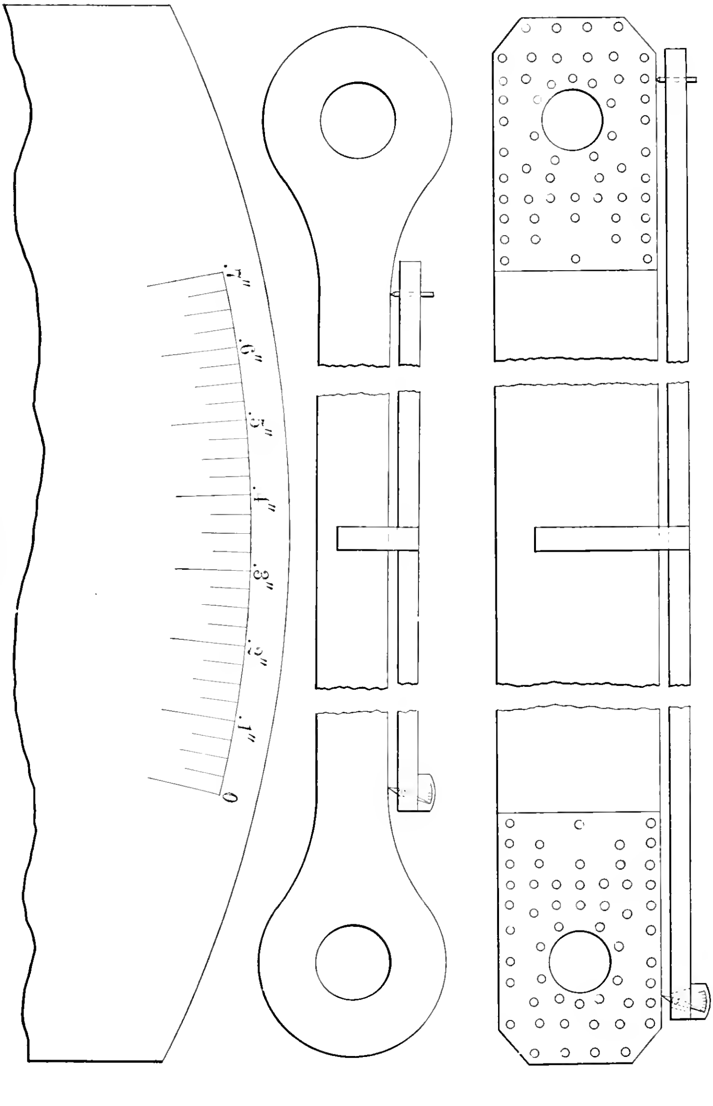
Fig. 2 illustrates an end of one of the 24-in. riveted bars, and shows the transmission of the strains by the rivets, as indicated by the cracking of the mill scale on the pin-plates, in a remarkably uniform manner.

All the riveted carbon-steel eye-bars broke off square, with granular fracture, little or no reduction of area, and elongations from 0.06 to 3.21% in the body of the bar. The nickel-steel riveted eye-bars. made of the softer heat of steel, containing 0.30% of carbon and 3.72% of nickel, broke with from 50 to 85% granular fractures, small reductions of area, and elongations from 2.17 to 3.11% in the body of the

*A report of these tests, together with a diagram showing the location of the test pieces in the plate, is filed in the Library of the Society.

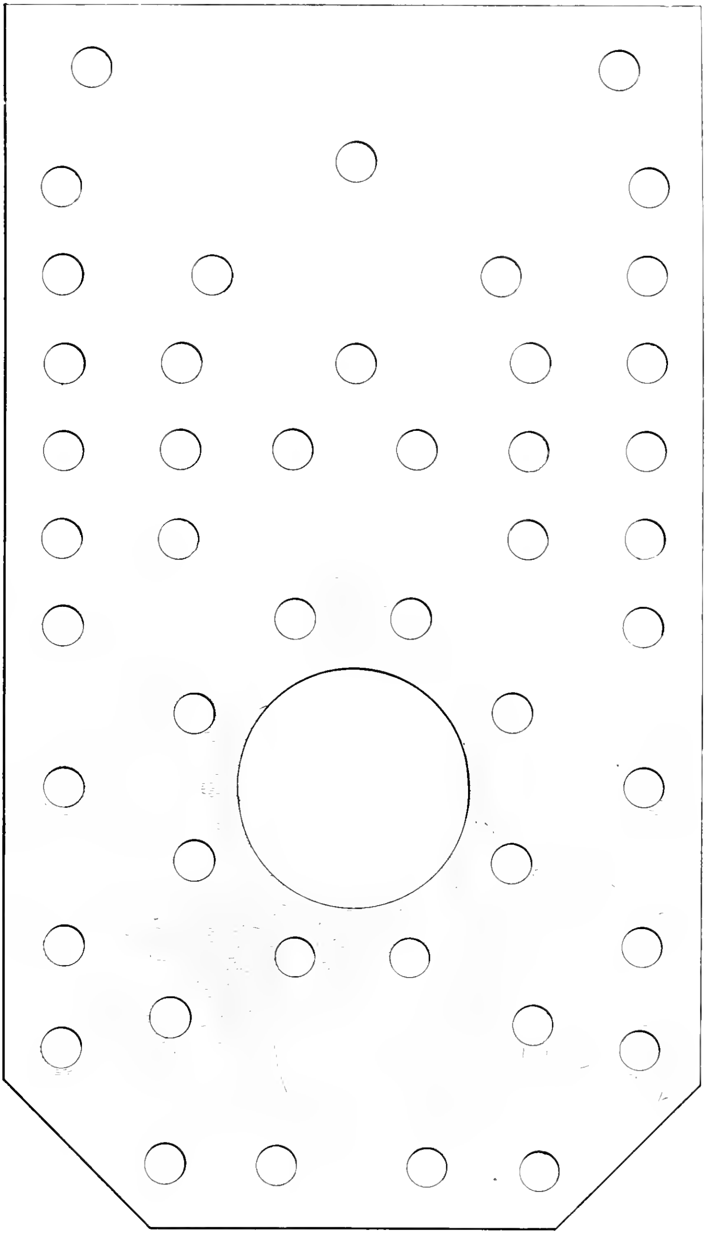
†Filed in the Library of the Society.

‡Full details of all tests, with extensometer readings, have been filed in the Library of the Society.



EXTENSOMETER. AS USED ON RIVETED AND FORGED EYE-BARS.

FULL-SIZED DETAIL
FIG. 1.



24" x 1" Bar with two Pin-Plates 24" x ½". Test reported June 30th, 1904.

Fig. 2.

bar; and all the harder steels, containing 0.477% of carbon and 3.56% of nickel and 0.424% of carbon and 3.57% of nickel, broke off square, with granular fractures, little or no reductions of area, and elongations from 0.78 to 2.39% in the body of the bar.*

Before these full-sized tests were completed, instructions were issued to the writer to make tests on nickel-steel forged eye-bars and to report on which bars he considered best suited for the structure. Twelve 14 by 2-in. bars, and six 16 by 2-in. bars were tested. Half of these bars were of acid open-hearth steel containing 0.33% of carbon and 3.72% of nickel; and half of basic open-hearth steel containing 0.36% of carbon and 3.40% of nickel. The same care was exercised in rolling these bars as in rolling the former bars. The eye-bars were all forged by The American Bridge Company, at Ambridge, Pa., and were tested at the works of The Phœnix Iron Company, Phœnixville, Pa., but some of the larger bars could not be broken as they were beyond the capacity of the machine.

Tension and bending tests were made from each end of each bar rolled.†

The writer reported on the results of all the tests made, advising the use of forged nickel-steel eye-bars.‡

Table 1 gives a summary of the results of all tests made on nickel-steel bars; the comparisons there given are between the net sections of the riveted bars and the forged bars. Table 2 gives the comparison between the gross section and the forged bars. For comparison, the average results of the specimen tests are given in each case.

Before any work was started on rolling the flats or forging the eye-bars for the structure, the following full-sized tests were called for, in order to test the uniformity of the material, methods of manufacture, and whether the upset head would develop the full strength of the bar:

Four heats of steel to be rolled:

3 full-sized eye-bars to be tested from the first heat
3 " " " " " " " second heat
2 " " " " " " " third heat
2 " " " " " " " fourth heat

*A blue-print and a report, giving the location and character of the fractures, is filed in the Library of the Society.

†Full details of all specimen tests and full-sized tests, showing extensometer readings, are filed in the Library of the Society.

‡Report filed in the Library of the Society.

Owing to the delay in completing the upsetting machine and the testing machine, however, about 2 000 tons of nickel-steel flats for eye-bars were rolled before these preliminary tests were made. As the contract specifications did not give any upper limit in tensile strength, the 2 000 tons of flats really consisted of two grades of steel, as the ultimate strengths in the sample tests ranged from 100 000 to as high as 126 000 lb. per sq. in.

The results of the ten full-sized tests of eye-bars from the first four heats were so irregular that they were considered experimental tests, and the manufacturer decided to test all the eye-bars which had been forged up to that time. These bars were annealed under varying conditions, using a pyrometer to determine the annealing temperatures, and were tested to destruction. Additional bars were also forged, annealed, and tested by the manufacturer before any eye-bars were submitted for test and inspection for the structure. In all, there were seventy-three of such bars tested.

In this preliminary work it was found that the results of tests on full-sized eye-bars, made from heats of steel of more than 115 000 lb. per sq. in. tensile in the sample, were so irregular that it was decided to make a full-sized test on an eye-bar from each of such heats of the 2 000 tons already rolled, and on all steel to be rolled, to make a full-sized test on each heat of more than 110 000 lb. per sq. in. tensile in the sample test, if the annealed sample test showed more than 100 000 lb. per sq. in.

As the work progressed, the following additional modifications were found advisable, in order to secure satisfactory eye-bars for the structure and prevent the use of brittle steel:*

Ultimate strength in unannealed specimen tests reduced to 95 000 lb. per sq. in.; all other provisions to stand as specified.

Ultimate strength in full-sized tests reduced from 85 000 to 80 000 lb. per sq. in.; all other provisions to stand as specified.

Eye-bars of heats of from 95 000 to 110 000 lb. per sq. in. in the unannealed specimen to be tested by lots or items, and 3% of such bars to be tested.

Test by heats all eye-bars made of steel of 113 000 lb. per sq. in. tensile, and over, if the annealed specimen test is over 100 000 lb.

Test by heats all eye-bars made of steel of 111 000 to 113 000 lb. per sq. in. tensile in the specimen test when carbon and manganese are very high and the annealed specimen test is over 100 000 lb.

*A report on these preliminary tests of 16-in. bars, and modifications desired, is filed in the Library of the Society.

Desired: Carbon not over 0.40, if manganese is 0.80 or over.
Desired: Carbon not over 0.45, if manganese is 0.70 or over.

In all the preliminary experimental eye-bars and those for the structure, the elastic limits were taken by the extensometer. Tests of eye-bars for the structure were made. The extensometer readings,* are grouped as follows:

Bars that met the requirements of the specifications;
†Bars that broke in the head, but were accepted under a modification of the specifications;
Bars that failed in elastic limit;
Bars from special high heats which broke in the head, with low elongation.

For comparison, some tests of carbon-steel eye-bars, showing the extensometer readings, are also filed in the Library of the Society.

All the nickel-steel eye-bars were tested on The American Bridge Company's large testing machine at their Ambridge Plant. The results on this machine are a little lower than those on the Government machine at the Watertown Arsenal.‡

The clause in the printed specification covering the annealing of eye-bars was not satisfactory for nickel steel, and the manufacturer determined the proper annealing temperature in the series of full-sized eye-bar tests referred to—the ten tests from the first four heats and the other full-sized tests made at that time. The pyrometer was used to determine temperatures in annealing all bars for the structure.

The bending tests on the bars showed that the wide full-thickness bends on unannealed specimens (180° $3T$) are a good check on the methods of rolling, and the full-sized bends of the pieces annealed with the eye-bars, a good check on the annealing.

The results of all the tests prove conclusively that a nickel steel of from 95 000 to 110 000 lb. per sq. in., tensile, in the unannealed specimen, and of 90 000 lb. per sq. in., minimum tensile, in the annealed specimen, would give eye-bars with a minimum elastic limit of 50 000 lb. per sq. in. in full-sized test and guard against brittle steel. The desired ultimate for full-sized tests was 90 000 lb. per sq. in., but 85 000 lb. would be accepted provided the eye-bars would show a minimum elastic limit of 50 000 lb. per sq. in.

*Filed in the Library of the Society.

†The Commissioner of Bridges modified the specifications, so that bars which broke in the head but developed an elastic limit of 51 000 lb. and an ultimate strength of 85 000 lb., with stretch of 6% in 18 ft., would be accepted.

‡The report of the standardization of the Ambridge machine is filed in the Library of the Society.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE OPERATION OF PASSENGER ELEVATORS.

Discussion.*

By Messrs. F. Lavis, Robert Brewster Stanton, and William J. Boucher.

F. Lavis, M. Am. Soc. C. E.—It might be interesting if Mr. Bolton would state why it is necessary to operate elevators on a schedule. The service does not seem to be comparable to railroad service, because, carrying out the simile, each elevator has its own track and its own station, and apparently each could be run entirely independently of all the others. Mr. Bolton gave an illustration of a case where there was a battery of six elevators, one having a much smaller door than the others and, consequently, taking longer to load and unload. He stated that the round-trip time of this slow elevator controlled the round-trip time of the other five. It would almost seem that this one might have been operated independently and the other five run on their own schedule, if a schedule was required. Mr. Lavis.

At times the speaker has occasion to visit the Park Row Building, where there is a large battery of elevators, and he has often noticed as many as three elevators on the ground floor at once, the waiting passengers being only allowed to enter one; when that was filled, those remaining might enter the second, and when that was filled, and started, the passengers still remaining might enter the third, by which time there were probably other cars waiting, and, at first sight, it would seem that time was being wasted, and that the cars might be

*This discussion (of the paper by Reginald Pelham Bolton, M. Am. Soc. C. E., printed in *Proceedings* for December, 1908), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Lavis. started and make their round trip each as quickly as possible and independently of the others.

No doubt there are very good reasons for the operation of elevators in rotation and as nearly as possible on schedule time, and of course regularity, as a general principle, is generally conceded to be a good thing; but it is not always safe for engineers to figure on generalities, and the speaker believes some definite reasons for the necessity of schedule operation would be of interest.

Mr. Stanton. ROBERT BREWSTER STANTON, M. AM. SOC. C. E.—To most engineers who have not made the question a study, it is, perhaps, somewhat surprising that the general problems of railway transportation enter so extensively into "The Operation of Passenger Elevators." The author has stated that different classes of passengers—the fat man, the lean man, the man who rushes in or out before the gates are fairly open, and the man who never "steps lively," as well as the woman with the hat 3 ft. in diameter—all affect the successful operation of passenger elevators, especially in tall office buildings.

It may not be out of place to ask the author whether, in his calculations, the question of classification of passengers has been taken into account in order to determine its effect upon the successful, and as far as the tenants and patrons of such buildings are concerned, the economical, operation of such elevators. Railways classify, not only their freight, but their passengers, though not, perhaps, on the lines above mentioned. The point to which it is desired to draw attention is this: In a large, centrally located building—one of the most expensive in the city—in which the speaker has had an office for the past six years, the employees of the building—scrubwomen with dirty, greasy buckets, bootblacks with large, besmeared boxes, and coal-heavers from the engine-room, with their clothes as black as soot—are allowed in every one of the ten passenger elevators of the building. This practice occasionally, in fact many times a day on some days, is a great inconvenience to the tenants and patrons of the building, as frequently they are obliged to wait for one or more trips before finding a car suitable for them to ride in to their offices. No objection is made to the employees—the men or the women—but, in rush hours, it is not pleasant to be squeezed in among the greasy buckets and blacking boxes, or against sooty jumpers, so that such practice, as with other things, hinders the regular and economical operation of the elevators.

Some months ago, the owners of the building referred to requested their tenants to make suggestions as to its better management. In reply, the speaker wrote and suggested that one elevator of the ten be set apart for employees. An order to this effect was issued, and for a few weeks carried out, but it was soon neglected, and all elevators were and are used as before. This is not on account of the want of courtesy of the elevator or hall men, but is traceable directly to the

Superintendent of the building, who, for some reason, refuses to carry out the orders of the owners. It would seem, therefore, that, in addition to those mentioned by the author, the disposition of the Superintendent is an item which should be taken into account as affecting, in the broad sense, the successful and economical operation of passenger elevators. Mr. Stanton.

William J. Boucher, Assoc. M. Am. Soc. C. E.—One of the largest and highest buildings in New York City is equipped with electric elevators, all on local service. There are ten cars, five of which run to the 26th floor and five to the 25th floor. They are of a type which was installed quite frequently ten years ago, but which has since been largely superseded. The elevators are not giving satisfaction, and the entire equipment needs rebuilding. The management, however, is confronted by the fact that they have several important tenants whom they cannot afford to lose, as they would if there were a complete shut-down. Further, the machines are located so that it is impossible for men to work among them while the others are in operation. There is seldom a day when at least one car is not out of commission—perhaps one for a part and another for the remainder of the day. Due to the overloading, which is inevitable during the rush hours, the "safety" frequently springs on the "down"; this causes inconvenience to the passengers, who have to leave the cars by means of a step-ladder, and the delay amounts to half an hour or more while inspection of the bearings and other parts is made. In the morning rush upward, the overloading is such that frequently several would-be passengers must leave the car before it can be started, and meanwhile the throngs are arriving and filling the hall, impatient to reach their offices. Mr. Boucher.

The operation of cars on "schedule," or consecutively, is no doubt the best system, its greatest feature being that the hall man knows which car to expect next, and can direct the waiting people where to stand. Something has been said of the hall man and his efficiency. A great deal depends on these men; they can give good or poor service; and be it said to their credit, they generally do all in their power to make matters go smoothly. They are often hampered by the operators on the cars, who, whether by reason of insufficient pay or pure negligence, are frequently careless, and run the cars to suit themselves by failing to stop for waiting passengers, this latter happening particularly at the top of the building.

The size or capacity of the cars has considerable to do with their efficiency, and one larger than its neighbors is a distinct drag on a system of elevators. The speaker learned to shun a car larger than its fellows, because it always started with so many passengers that their disembarkation delayed it so that it arrived at the upper floors later than the car which followed it from the ground floor.

A few years ago, a test of these elevators was conducted as a thesis

Mr. Boucher. test by students of Stevens Institute of Technology, and the following figures are taken from that test:

Offices per floor (various sizes)...............	39
Height, from ground to 26th floor............	309 ft.
Distance between floors (generally)...........	12 "
Floor area of cars........................	36 to 44 sq. ft.
Average number of round trips per hour......	16
Average number of passengers over entire trip..	5.5

This last figure, however, is subject to great fluctuation, as cars frequently start with as many as fifteen passengers going up, and arrive at the top floor with one or two passengers. The tests showed:

Starting up, from ground, per hour.	126 to 178 passengers.
Arriving at 26th floor.............	19 to 22 passengers.

Due to the peculiar system of roping these cars and counter-weights, there is a great length of cable, and unless the operator is particularly skillful, there is a distinct and disagreeable "lurch" or "surge" at the stop, and many women passengers are made faint by the sensation.

Mention has been made of the newer electric installations, but it seems to the speaker as though the plunger hydraulic is giving the most satisfactory service. Of course, there is some question as to the permanency of the installation tubes placed in the ground, and wet inside and outside, and it would be very interesting if Mr. Bolton would contribute from his knowledge the latest ideas on the subject.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE BONDING OF NEW TO OLD CONCRETE.

Discussion.*

By Messrs. H. R. Burroughs, Theodore Belzner, and James L. Davis.

H. R. Burroughs, Jun. Am. Soc. C. E. (by letter).—The writer was much interested in Mr. Goodrich's paper, as it recalled a test which had come to his notice and may be worthy of mention. Mr. Burroughs.

The writer was required to make an inspection of a reinforced concrete building, the report having been prompted by the fact, that upon stripping the forms, the concrete was found to be porous, with many visible pockets. After considerable investigation, it was concluded that it would be necessary to remove the porous parts and refill the places thus made vacant. The expedient of bonding new concrete to old was resorted to, as the defective places had already been poured several weeks. No apprehension, however, was displayed in recommending this method of procedure, which was undertaken and, to all appearances, carried out with success and satisfaction.

In view of this undertaking and incident thereto, the Superintendent in charge conceived the idea of making a rough bonding test, more to satisfy his personal curiosity than to arrive at any conclusion. He selected a barrel and poured into it the regular concrete mixture, as used on the work, until it was one-third filled; this was permitted to set for seven days, when the second third of the capacity of the barrel was poured; this also was permitted to set for seven days, when the last third was poured, filling the barrel. The barrel was then set

* This discussion (of the paper by E. P. Goodrich, M. Am. Soc. C. E., printed in *Proceedings* for January, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Burroughs. aside for two weeks, when the staves were stripped from the concrete, exposing the concrete mass. It was noted that the lines of cleavage were scarcely visible, the whole exhibiting an appearance of continuity and of being monolithic.

It was then attempted to break this concrete core by well-directed blows from a sledge. It required several severe and direct blows, however, to start a break, the ultimate rupture occurring, as might be expected, on a line of junction between two zones of different times of pouring.

The bond or cohesion displayed, however, seemed remarkable, inasmuch as no attempt had been made to effect a union of the parts, the surfaces not having been washed or cleaned.

Mr. Belzner. THEODORE BELZNER, JUN. AM. SOC. C. E. (by letter).—The writer has read with much interest the description of these laboratory experiments on the bonding of new to old concrete, and also the published deductions from tests made by Messrs. Perry and Mesnager.

There are many methods for bonding concrete, but the writer will confine his discussion to that in which the strength depends on the degree of adhesion of concrete to concrete.

Excellent results in bonding can be secured in the field by conscientious treatment and intelligent supervision, although such results cannot be compared with those of the laboratory, as, in the latter case, the experiments are made under conditions which are practically impossible to secure in the field.

Laboratory experiments have determined that roughening the old concrete surfaces and applying a layer of cement paste have given the best results. The writer has used this method on construction work in the New York Subway and on other engineering work. He has had occasion to examine concrete jack-arches, walls, etc., in which great care was exercised in the bonding, and while these arches appeared to be monolithic for a year or more after their construction, upon close examination, they revealed fine hair-line cracks on the surface. These cracks could be readily traced at the junction of the work of one day with that of another, indicating a break between the bond. On many other occasions he has observed breaks in the bond of concrete constructed under less careful supervision, the joints of which could easily be traced between the layers of concrete. This was proved conclusively by examining concrete which had been demolished and in which the joints had been separated. The writer also recalls several instances when engaged in locating leaks in steel-concrete construction and in jack- and roof-arches,* where surface water had percolated through the horizontal joints. These investigations have convinced him that perfect joints in concrete are a fallacy, experience having taught him that the different days' work

* *Transactions*, Am. Soc. C. E., Vol. LI, p. 130.

can always be traced, despite any precautions which may have been taken. It is his opinion that investigators should continue their laboratory experiments along this line, but before acid and other treatments are permanently adopted in the field (even though good results have been obtained on a small scale), the present methods should be continued (roughening the old concrete surfaces and applying a cement paste) until acid treatments, etc., have proved positively their practical efficiency. Mr. Belzner.

The only possible way of securing a perfect monolithic structure is by carrying on the work continuously, day and night, until finished. This can be done, at times, on small structures, but is practically impossible with structures of magnitude.

JAMES L. DAVIS, ASSOC. M. AM. SOC. C. E.—The speaker has made a small series of bonding tests for the Board of Water Supply of the City of New York, under the direction of Ernst F. Jonson, Assoc. M. Am. Soc. C. E., Engineer Inspector. Mr. Davis.

The purpose of the tests was to determine the efficiency of a proprietary bonding compound. The test specimens were beams 6 by 6 in. in cross-section, and 30 in. long. The concrete was a 1:3:5 mixture, with ordinary sand and gravel aggregate. The tests were conducted as follows:

Three beams were broken by central loading on a 26-in. span at the age of 28 days. All broke very nearly in the middle, and gave ordinary strength values. Each half of the three beams was put into a wooden mould similar to those in which they were cast, leaving one-half of the mould empty. New halves were then cast, restoring the beams to their original dimensions, making six specimens. Before casting the new halves, the fractured end surfaces of the old beams were wetted and covered with a cement grout of as heavy consistency as could be applied with a brush. In making the grout for three of the beams, the bonding compound was dissolved in the mixing water as directed by the makers. For the other three, the blanks, the grout contained none of the bonding compound. Care was taken to work the fresh concrete thoroughly in the vicinity of the joint, using a bricklayer's trowel. This expelled the air and insured contact with the old surface.

After 28 days, the beams were again broken in the same manner as before. The breaks occurred in the new concrete about $\frac{3}{4}$ in. from the joints, in one beam touching the joint at one point. There was no difference in the results where the bonding compound was used and where it was not. The result in every case was a perfect bond within the limit of the strength of 28-day concrete.

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PAPERS AND DISCUSSIONS

THE ACTION OF FROST ON CEMENT AND CEMENT MORTAR, TOGETHER WITH OTHER EXPERIMENTS ON THESE MATERIALS.

Discussion.*

By Messrs. William Mayo Venable, Herbert W. Hatton, and John C. Wait.

Mr. Venable. William Mayo Venable, M. Am. Soc. C. E. (by letter).—The writer is much interested in this valuable paper, having had experience with the effect of frost on concrete, and the effect of using salt water in mixing concrete.

In the construction of the Long Key Viaduct, on the Florida East Coast Railway, the concrete was at first mixed with fresh water only; but, owing to the expense of supplying fresh water for this purpose, so many miles from any natural source of supply, salt water was finally used, it being believed that it would not have an appreciably injurious effect. On pier work it was customary to allow the coffer-dams to fill with salt water before the removal of the forms, and in less than a day after the placing of the concrete; and it is interesting to note that the tests set forth in *Experiment F* and *Experiment G* indicate that no appreciable difference in the strength of concrete mixed with fresh water and that mixed with sea water need be expected. The writer's reason, however, for preferring fresh water was doubt as to the action of the salt water on steel reinforcing rods.

The conclusion that mixing cement mortar with warm water has a

* This discussion (of the paper by Ernest R. Matthews, Esq., and James Watson, Esq., printed in *Proceedings* for January, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

permanently injurious effect, if not requiring some modification, is most important, as this is far the easiest way by which the temperature of materials for concreting in freezing weather may be raised sufficiently to cause the concrete to set. On all the sections of the sewer system now under construction in Louisville, Ky., concreting is permitted when the temperature is above 24° Fahr., but the sand, gravel and water must be separately heated before mixing. The concrete is hardly likely to be frozen after being deposited, owing to its protected location. If any person has investigated the subject of the effect of temperature, above freezing, upon the strength of concrete, and the rapidity with which the full strength is acquired, the data obtained ought to be communicated. It may be that the heating of the ingredients separately is as injurious as the separate heating of the water; or that heating the water is injurious only when it raises the temperature of the resultant mixture above a certain point. The practice of heating concrete materials, generally the water, to remove frost in gravel and sand, and thereby obtain concrete at about 60° in the forms, thus securing sufficiently rapid initial set, is quite general, and some method of heating is necessary if concreting is to be done at all when the outside temperature is below freezing. Mr. Venable.

Can any person furnish the results of a series of tests upon mortar mixed with warm water, showing to what temperature the water, or the mortar, may be raised without injury to the ultimate strength to be acquired? The writer has frequently heated water in mixing concrete to more than 80° Fahr., producing concrete at 54° or higher as it left the mixer, and has never discovered any weakness in such concrete; and if the heating of the water alone is not injurious, below a certain temperature, it is much better than heating the sand and gravel separately, because it is simpler and more economical than any other method.

Herbert W. Hatton, Jun. Am. Soc. C. E. (by letter).—In reviewing this very excellent paper, the constructing or field engineer at once seeks to make comparisons between the respective tests recorded and actual construction under similar conditions; for after all it is the results obtained in the field under alternate conditions of frost and thaw which are of paramount importance. Mr. Hatton.

The conditions under which concrete is tested in a laboratory are vastly different from those encountered in the field, for example, suppose 100 cu. yd. of concrete to be mixed between temperatures varying from 28 to 40° Fahr.—and this is frequently the case—the main differences between concrete mixed in the laboratory and concrete mixed on the work will be:

First.—The cement used in one case has been practically kept in cold storage, and its actions in setting are retarded, whereas that used in the testing laboratory is kept practically at 60° Fahr., summer heat,

Mr. Hatton. hence crystallization begins at once upon the making of the test pieces.

Second.—The water used in mixing the concrete is usually very cold, or it is heated until it is quite hot, and, in either case, setting is greatly retarded.

Third.—The ingredients forming the concrete are either cold or they may be heated; in the one case they may be full of frost, and, in the other they may be so hot as to absorb readily large quantities of water from the cement and needed by it in crystallization.

Fourth.—Concrete mixed when the temperature is 29° Fahr., or thereabout, will get neither its initial set nor hard set as quickly as that mixed when the temperature is 60° Fahr., nor is it possible in all cases to maintain the surrounding atmosphere at a temperature of 60° Fahr. for 24 hours, except at considerable expense.

It seems to the writer that the result of the action of frost and thaw on concrete can best be obtained from actual cases in the field. It is true that practical tests, at a great deal more expense, can readily be made under actual working conditions, and possibly such tests could be made by the committee which has received the Government appropriation, set aside for the past year or more, for testing concrete beams, etc. This paper, however, is quite a valuable asset to the literature on this subject.

An actual case of recent date, where concrete was placed when the temperature varied many degrees, may be of interest. During the early part of the winter of 1908, the writer was engaged upon the construction of a large dam at Berwick, Pa., and incidental thereto, the spillway of an old dam had to be torn out and replaced. Cold weather came at a time when the entire paving, concrete apron, etc., of the spillway, had been removed, and it was necessary to repave and concrete before the spring freshets came; this necessitated the placing of the concrete covering and apron at the toe regardless of the weather conditions.

The spillway was 60 ft. wide and about 55 ft. long, with a slope of 1½ to 1. The thickness of the concrete was 7 in. The number of square feet of concrete placed each day was too great to heat readily. During the mixing of the concrete, the temperature ranged from 28 to 40° Fahr., much of it being mixed at the lower temperature. The stone was full of frost, as was the sand, and the water came directly from the dam, ice being broken in order to get it. The cement was stored in a shed, the temperature within which was the same as the outside atmosphere.

In all cases the sand was heated by a fire, kept constantly burning within a 16-in. cast-iron pipe, the sand being spread over and around the pipe. The water was also heated until it steamed. In using the water, the practice was to empty two buckets of ice water into the large

kettle each time two buckets of water were taken therefrom for mixing concrete, and in this way the temperature of the water used in mixing was reduced so that it had no harmful effect on the concrete. Several barrels of cold water were kept close at hand for this purpose. Mr. Hatton.

The heated sand and the cold cement were first mixed thoroughly, and the stones were placed thereon, then the warm water was poured over the stones, and in this way the temperature of the water was further reduced, so that on the completion of the mixing the temperature of the mixture rarely exceeded 50° Fahr.

In no case was the mixing carried on when the temperature was below 27° Fahr., and whenever the thermometer stood below freezing the concrete was covered as fast as it was placed.

In order to ascertain how much covering of manure was necessary to insure the non-freezing of the concrete under varying conditions of temperature, the following experiment was made.

Small quantities of concrete were mixed as described, and, when placed, were immediately covered with cement bags, or with tarpaulins, upon which hot manure was spread. The temperature on two occasions dropped to 10° Fahr., but, with a covering of manure 8 in. thick, the concrete was in no case affected. Coverings 4 and 5 in. thick were found insufficient, as this small body did not contain heat enough in itself to continue to generate heat from the phosphorus, ammonia, etc., therein contained, and in two days' time the frost practically went through it.

Almost every night the temperature dropped to 24° Fahr., frequently went down to 15 or 16°, and on two occasions down to 10° Fahr., but the covering, which was 8 in. thick, on top of cement bags or tarpaulins, proved adequate, except when a rain occurred, and the covering got wet, and then even a thickness of 8 in. was insufficient, as the water dripped down through the manure, and when the temperature dropped, as it usually did after each rain, the frost went through the covering, forming a heavy white frost directly on top of the tarpaulin and also reaching the concrete. Therefore, whenever a rain occurred, in case the concrete had not already obtained its hard set, it was necessary to take off the old manure and replace it with a fresh covering.

The manure was obtained from the contractor's mules, which were kept within a few hundred yards of the operation, thus the only cost connected with the covering of the concrete was the hauling and the spreading. The farmers in the vicinity were eager to collect the manure and haul it away without cost, after it had been used for this purpose.

Nazareth cement was used on the work, and was very slow setting. In some cases, when mixed in concrete, it did not obtain its hard set in from 30 to 40 hours.

Mr. Hatton. The following method was used to determine whether, in any instance, the frost had worked beneath the covering: Before placing the tarpaulin several quarts of water were poured over the concrete where there was a slight hollow place or indentation, and this water was added to, until the concrete had taken up all it would hold. It was not necessary to add more water after the concrete had obtained its hard set, and in no case was there the slightest indication of freezing during a daily examination for five consecutive days.

In a couple of instances the writer had the manure covering removed within four days, but there was still considerable moisture in the concrete, and on the surface, due to the manure. This moisture was soon frozen, and after a subsequent thaw, the top surface could be scraped or peeled off.

The practice thereafter was to leave the covering on for at least a week or ten days. When it was removed, the concrete was in first-class condition, and apparently continued to harden uniformly. In using pick and sledge on this concrete, the writer found that, as far as he could judge, it had set just as well as if it had been mixed under the most favorable conditions.

This is directly in line with the statements of Messrs. Matthews and Watson, to the effect that if a cement or concrete obtains its hard or final set before being exposed to frost, it will, upon exposure, continue to crystallize and harden.

The hot water with the frosty stone had a tendency to create a normal condition under which the cement suffered no injury.

Considerable care should be taken, however, to protect the manure from becoming wet during rain storms. If possible, it should be covered with tarpaulins, or with planks arranged in such a manner as to exclude the water. This is a practical and yet a very inexpensive way to protect concrete in the open where a low flat surface is presented.

From this experience, the writer has no hesitation in saying that, in his opinion, if concrete is properly mixed and protected until it has obtained its final or hard set, a frosty atmosphere, with the temperature in the neighborhood of 28 or 29° Fahr., has no permanent or detrimental effect upon it.

Mr. Wait. JOHN C. WAIT, M. AM. SOC. C. E.—Some twenty-four years ago the question of the use of cement in freezing weather was discussed before this Society, and the speaker supposed it had been settled.

In South Chicago, twenty-four years ago, the temperature being from 10 to 24° above zero, the speaker put in foundations for large and heavy car shops, and then built the walls, all being laid with Portland cement mortar. In the following summer, having occasion to dig them up to put in sewer pipes, they were found to be perfectly set and strong. The speaker agrees with Mr. Stern, who has said that, if concrete

foundations can be buried or enclosed so as to let the frost come out at its leisure, the concrete will set in time. Mr. Wait.

The great power station and pulp mills at Rumford Falls were put in during extremely cold weather, but the speaker has never heard that there was any trouble with them. The foundations carry very heavy machinery, and are considered a good piece of work.

It would seem that the subject of heating is one that is much feared, and without reason. In the month of November the speaker visited Youngstown, Ohio, where he saw concrete mixed with Atlas cement and with slag cement and poured into moulds that were steam-jacketed and heated, so that the mixture boiled and steamed. In from 5 to 7 min. the concrete was taken out of the moulds and handled in the form of blocks, the walls and partitions being from $\frac{1}{2}$ to $\frac{5}{8}$ in. thick. These concrete blocks were put on cars and run into steam-rooms, where they were allowed to set. The speaker never saw any better ultimate result with concrete than these blocks, and they are now used extensively in Youngstown for the construction of houses.

Heating the water, or heating the sand or the stone does not hurt concrete materially unless it is superheated, and certainly if it is not heated above the boiling point.

It would be enlightening to hear an expression of opinion from those who may have seen this work at Youngstown, or who have had experience with heated concrete.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOSEPH MARSHALL GRAHAM, M. Am. Soc. C. E.*

DIED FEBRUARY 3D, 1909.

Joseph Marshall Graham was born at Crawfordsville, Indiana, in 1850. He received his education at the Kentucky State University, at Lexington. He began his railroad career, in 1873, as Assistant Engineer on the Grayville and Mattoon Railroad, now a part of the Illinois Central. From 1874 to 1875 he was Assistant Engineer on the Bedford, Springville, Owensburg and Bloomfield Railroad; and, from 1875 to 1876, he served as Chief Engineer of the same road. After that time he held the following positions:

From 1876 to 1881, Chief Engineer of the Danville, Olney and Ohio River Railroad; from October, 1881, to April, 1882, Chief Engineer of the Chicago, Texas and Mexican Central Railway; from April, 1882, to April, 1883, General Superintendent of the Danville, Olney and Ohio River Railroad; from April, 1883, to October, 1888, Superintendent of the Dakota Division, Northern Pacific Railroad; from October, 1888, to October, 1890, General Manager of the Northern Pacific and Manitoba Railway of the Northern Pacific System; from October 1st, 1890, to July, 1891, Assistant General Superintendent of the Northern Pacific Lines East of Livingston, Montana; from September, 1891, to January 1st, 1898, Superintendent of the Ohio and Midland Divisions, Baltimore and Ohio Railroad, at Newark, Ohio; from January 1st, 1898, to July, 1899, General Superintendent of the Trans-Ohio Division of the same road; from July 1st, 1899, to February, 1904, Chief Engineer of the Baltimore and Ohio Railroad at Baltimore, Maryland; from February, 1904, to 1908, Vice-President of the Erie Railroad; and, in 1908, he was made Vice-President of the New York, Susquehanna and Western Railroad, which position he held at the time of his death.

Mr. Graham was one of the pioneers in advocating and carrying out important grade-reduction work, to lessen train-mile cost; and while Superintendent of the Baltimore and Ohio Railroad, in 1893, he commenced the modification of the grades on the Lines West of the Ohio River. During the time he was Chief Engineer of that road, surveys were made, and a low-grade line was located from Baltimore to Chicago, a considerable portion of this line being constructed under his direction, viz., from Baltimore to Brunswick, Maryland; various grade-reduction improvements were made between Harpers

*Memoir prepared by Francis Lee Stuart, M. Am. Soc. C. E.

Ferry and Cumberland; a freight line was constructed around Cumberland, as well as a line from Youngstown to Akron; grade reductions were made on the Cleveland, Lorain and Wheeling Railroad, together with various grade reductions in connection with double-tracking the Lines West of the Ohio River.

When it was determined by the Directors of the Erie Railroad to relocate certain portions of its lines, in order to make it a low-grade line between Chicago and New York, Mr. Graham was called from the Baltimore and Ohio Railroad to take charge of the work as Vice-President of the Erie Railroad; and it was under his direction that plans and surveys were made for a line from New York to Cleveland with a ruling gradient of 0.2% east and 0.3% west, and the Erie and Jersey Railroad and the Genesee River Railroad, which are parts of this low-grade line, were constructed.

During 1908, Mr. Graham had relinquished some of the executive duties of his office to engage as Consulting Engineer in several large enterprises in different parts of the United States.

Mr. Graham was a man of large ideas, holding broad views of the railroad questions of his time, and his habits of mind, mature judgment, and sound conclusions on all practical topics of railway economics, made him an invaluable addition to any organization.

In 1906, the Kentucky State University conferred on him the degree of Doctor of Engineering.

In 1880 Mr. Graham married Miss Evelyn Norton, daughter of the Reverend Albert Norton, of Cleveland, Ohio, who survives him.

Mr. Graham was elected a Member of the American Society of Civil Engineers, on April 4th, 1900.

CLINTON GLENCAIRN WELLS, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 16TH, 1908.

Clinton Glencairn Wells was born in Galveston, Texas, on December 13th, 1874. He attended school in Galveston, and in Lawrenceville, New Jersey. He was graduated from Princeton University, in Civil Engineering, in 1898. His first employment in his chosen profession was with the Jersey City Water Supply Company of Boonton, New Jersey, in 1899, in the capacity of Assistant to the Engineer in charge of a party making preliminary surveys and location of the works to carry out a $7 695 000 contract with Jersey City.

On May 1st, 1900, Mr. Wells accepted a position with the Electrical Commission of Baltimore City, in the capacity of Engineer in Charge

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

of Distribution, on an underground conduit system for the reception of wires of all classes. In October, 1901, he left the Electrical Commission to accept a position with the Maryland and Pennsylvania Railway Company, as Assistant Engineer in Charge of Reconstruction Work in Western Maryland, straightening the line and laying out new curves.

On June 10th, 1902, Mr. Wells was elected City Engineer of Galveston, Texas, in which capacity he served faithfully until May, 1907, when, by reason of ill health, he was forced to resign. At the time Mr. Wells was elected the city was just beginning to recover from the great calamity of 1900, and his five years of service might be termed the period of its rehabilitation; most of the improvements, both public and private, were due to his skill and wisdom, and were made under his immediate supervision.

In December, 1904, Mr. Wells was married to Miss Josephine Kenison of Galveston, Texas, who with their only child, Clinton G. Wells, Jr., and one sister, Miss Hetty L. Wells, residing in Baltimore, Maryland, survives him.

One of Mr. Wells' friends and associates, Mr. W. A. Nicholson, in a recent letter, feelingly expresses sentiments held by all who really knew him:

"No words can be written that will fully express the purity of purpose that ever actuated this noble hearted boy. It can be truly said, that the light which filled his manly breast would permit him to 'sit in the center of night and enjoy bright day' and why Fate should deal so unkindly with him, and why he should die so young in the very spring-time of his manhood, is to me a mystery."

Mr. Wells was elected a Junior of the American Society of Civil Engineers on March 5th, 1901, and an Associate Member on March 1st, 1905.

OLIN McCLINTOCK BOYLE, Jr., Jun. Am. Soc. C. E.*

DIED AUGUST 19TH, 1908.

Olin McClintock Boyle, Jr., was born near San José, California, on July 5th, 1886. He was graduated from the grammar grades of the Willow Glen School, and entered the Santa Clara High School in September, 1898. Six months later his family removed to San Francisco, where he entered the Lowell High School. During the summer vacation of 1901 he was induced by the city editor of the *San Francisco Examiner* to become a writer on that paper. His short newspaper career was unusually promising. He rapidly rose to some-

*Memoir prepared by Charles Derleth, Jr., M. Am. Soc. C. E.

thing more than a mere reporter. Later, he accepted the sporting editorship of the *San Francisco Evening Post*, and held it until October, 1902, when he re-entered the Lowell High School, from which he was graduated in December of that year. He was admitted to the University of California in January, 1903, and was graduated from its College of Civil Engineering in December, 1906, having completed most satisfactorily the usual four-year course.

Immediately after his graduation Mr. Boyle went to the Greenwater Copper District of Inyo County, California, as Assistant to Mr. Alfred J. Cleary, remaining until the camp was closed. Returning to San Francisco, in May, 1907, he received an appointment as Instructor in the Santa Cruz Summer School of Surveying of the University of California. At the close of this session, he was employed for a short time by the Modesto Irrigation District Company, but left their employ to accept a position as Engineer with the Associated Pipe Line Company. Early in 1908 he became connected with the Union Construction Company, with headquarters in Calaveras County, California, and was in the employ of this company when he met his untimely death on August 19th, 1908.

In his short life Mr. Boyle gave promise of an unusually active and successful career. He was a man of great physical strength, and was a most industrious worker. He had a charming personality. He was constantly on the alert to be of service to others and to make others happy.

Less than two years elapsed between his graduation from his University and his death, but in that short time he gave numerous evidences of his love for his Alma Mater, and showed by his acts that he fully realized his indebtedness to the institution which trained him.

His early experiences in newspaper work had given him a taste for reading, and he was unusually well informed. In the year preceding his death, he contributed a number of short articles to the *California Journal of Technology*, wherein he showed promise as an engineering author.

It is fitting that the *Transactions* of the Society, which represents most broadly the profession he revered, should contain a statement of his life and work so that those who were associated with him may be reminded of his substantial character and loving personality.

Mr. Boyle was elected a Junior of the American Society of Civil Engineers on March 5th, 1907.

William P. Morse

PROCEEDINGS

OF THE

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OF

CIVIL ENGINEERS

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

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This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

March 17th, 1909.—The meeting was called to order at 8.30 P. M.; Vice-President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 155 members, and 42 guests.

A paper by L. R. Gifford, Assoc. M. Am. Soc. C. E., entitled "Steel Sheeting and Sheet-Piling," was presented by the author.

Communications on the subject from Messrs. Charles W. Sherman and C. C. Conkling were read by the Secretary; and the paper was discussed orally by Messrs. E. P. Goodrich, F. W. Skinner, J. C. Meem, R. B. Woodworth, and the author.

The Secretary anounced the following deaths:

Howard Breen, elected Member April 4th, 1888; died February 14th, 1909.

Charles Herbert Deans, elected Junior December 3d, 1890; Associate Member May 6th, 1896; died March 7th, 1909.

Adjourned.

April 7th, 1909.—The meeting was called to order at 8.30 P. M.; President Onward Bates, in the chair; Chas. Warren Hunt, Secretary; and present, also, 111 members and 24 guests.

The minutes of the meetings of February 17th and March 3d, 1909, were approved as printed in *Proceedings* for March, 1909.

A paper by E. E. Howard, Assoc. M. Am. Soc. C. E., entitled "The Sixth Street Viaduct, Kansas City," was presented by the Secretary, and illustrated with lantern slides.

Written communications on the paper, from Messrs. Victor H. Cochrane and Daniel Bontecou, were read by the Secretary, and the subject was discussed orally by Messrs. G. H. Pegram and O. E. Mogensen.

The Secretary announced the election of the following candidates by the Board of Direction on April 6th, 1909:

As Members.

William Pope Anderson, Cincinnati, Ohio.
George Parker Bard, Boonton, N. J.
Charles Adrian Hammond, Mount Vernon, N. Y.
John Byers Holbrook, New York City.
Thomas John Morrison, Rochester, N. Y.
William Welcome Peabody, White Plains, N. Y.
George Perrine, New York City.
Frederick George Ray, Detroit, Mich.
Walter Britton Rittenhouse, Duluth, Minn.
Walter William Schlecht, Washington, D. C.
David Charles Serber, New York City.
Howard Eveleth Stevens, St. Paul, Minn.
Carlton Carpenter Witt, Sioux Falls, S. Dak.

As Associate Members.

Alfred Curtis Ackenheil, Aspinwall, Pa.
George Wishart Butz, Schuylkill Haven, Pa.
Frederick Charles Carstarphen, Denver, Colo.
James William Elliott, Burlington, Vt.
Charles Needham Forrest, Maurer, N. J.
Felder Furlow, Manãos, Brazil.
James Cowan Green, Albany, N. Y.
Jesse Warren Ijams, Albany, N. Y.
Maro Johnson, Chicago, Ill.
Samuel Reynolds Jones, New York City.
Arthur Stephen Lewis, Chicago, Ill.
Carl Bertram Lindholm, Russell, Mass.
Oscar Charles Merrill, Berkeley, Cal.
Fred Burgess Nelson, North Olmsted, Ohio.

George Bruce Palmer, Pittsburg, Pa.
David Thomas Pitkethly, Brooklyn, N. Y.
Frederic Adams Reimer, East Orange, N. J.
Thomas Riggs, Jr., Washington, D. C.
Donald Hubbard Sawyer, Seattle, Wash.
Thomas Arthur Smith, Jacksonville, Fla.
Elihu William Stevens, Hackensack, N. J.
Harold Lyell Stevens, Houston, Tex.
Robert Summers Stockton, Glendive, Mont.
Omner Jay Turley, Aztec, N. Mex.
Edmund Boyd Ulrich, Reading, Pa.
John Henry Leon Vogt, Louisville, Ky.
Adelbert Alonzo Weiland, Pueblo, Colo.
Lewis Maxwell Young, East Boston, Mass.

As Associate.

Frederick Eugene Townsend, New York City.

As Juniors.

Lowrey Wallace Anderson, Dallas, Tex.
Willard Townshend Chevalier, New York City.
William Franklin Collar, Princeton, Mich.
Herbert McMillen Dibert, Troy, N. Y.
James Calvin Foss, Jr., Wailuku, Hawaii.
Alvin Leroy Gilmore, New York City.
Chester Mason Gould, Cold Spring on Hudson, N. Y.
Edward Parmelee Hamilton, Havana, Cuba.
Benjamin Earl Jones, Washington, D. C.
Harold Vincent Joslin, Norfolk, Va.
Frank Edward Reed, Carmel, N. Y.
Patricio Andres Suarez y Cordoves, Santiago de Cuba, Cuba.
Charles Leopold Walker, Ithaca, N. Y.
James Madison Warner, Chicago, Ill.
George Lansing Wentworth, New York City.

The Secretary announced the transfer of the following candidates by the Board of Direction on April 6th, 1909:

From Associate Member to Member.

Frederick Auryansen, Jamaica, N. Y.
George Herbert Brazer, Boston, Mass.
Myron Samuel Falk, New York City.
Julian Hastings Granbery, New York City.
Joseph Jacobs, Portland, Ore.
Edwin Seton Jarrett, New York City.

Oscar Francis Lackey, Baltimore, Md.
Henry Bennett Machen, New York City.
John Wiley Miles, City of Mexico, Mex.
Henry Lewis Oestreich, New York City.
Howard Flanders Taylor, Kansas City, Mo.
Albert Lowry Webster, New York City.

From Junior to Associate Member.

Alexander Seymour Ackerman, Sandwich, Mass.
William Godfrey Arn, Corinth, Miss.
Collingwood Bruce Brown, Jr., Montreal, Canada.
Henry Avery Campbell, San Francisco, Cal.
Robert Edward Carrick, New York City.
Edward Christian Dicke, St. Louis, Mo.
Malcolm Elliott, Culebra, Canal Zone, Panama.
Thomas Lilly Fountain, Houston, Tex.
Thomas Hagen Gronwall, Pittsburg, Pa.
Alexander Sylvester Hamill, Jersey City, N. J.
William Heer, Jr., Tuscaloosa, Ala.
Leslie McHarg, New York City.
Charles Wesley Malcolm, Urbana, Ill.
Stanley Alfred Miller, Azua, Santo Domingo.
Frederick Horace Tibbetts, Berkeley, Cal.
John Hough Wickersham, Lancaster, Pa.

From Junior to Associate.

Henry Rogers Codwise, Brooklyn, N. Y.

The Secretary announced the following deaths:

James Eager Willard, elected Member May 1st, 1889; died March 8th, 1909.

Julius William Schaub, elected Junior November 5th, 1884; Member October 6th, 1886; died March 30th, 1909.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

March 2d, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Brackett, Christie, Churchill, Hazen, Knap, Pegram, Schneider, Stearns, Swensson, Thompson, and Tillson.

Ballots for Membership were canvassed, and 11 Members, 13 Associate Members, 1 Associate, and 12 Juniors were elected, and 3 Juniors were transferred to the grade of Associate Member.

The Constitution adopted by the Colorado Association of Members of the American Society of Civil Engineers was approved.

The Secretary was authorized to issue a circular calling attention to a meeting for the consideration of the subject of the Conservation of Natural Resources to be held March 24th, 1909, at the Engineering Building.

Eight Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

April 6th, 1909.—President Bates in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Andrews, Brackett, Endicott, Kittredge, Noble, Pegram, Stuart, Swain, Swensson, Thompson, Tillson, and Williams.

Ballots for Membership were canvassed, and 13 Members, 28 Associate Members, 1 Associate, and 15 Juniors were elected, 16 Juniors were transferred to the grade of Associate Member, and 1 Junior was transferred to the grade of Associate.

The following resolutions were adopted:

Whereas, the accommodations available at the place of the coming Annual Convention are unlikely to exceed those needed for our own members, it is resolved that the Society do not invite any other National Society to participate in the said Convention.

Resolved, that members be informed that it is desirable that only those of their immediate family should be included in the attendance.

Resolved, that the Secretary be instructed, at the Annual Convention, not to enter on the list of those in attendance as guests any one who is not a member of the immediate family of a member of the Society.

The Secretary was authorized to issue to all members of the Society a pamphlet containing the addresses delivered at the joint meeting of Engineers, held March 24th, on the Conservation of Natural Resources.

Twelve Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

MEETINGS

Wednesday, May 5th, 1909.—8.30 P. M.—Two papers will be presented for discussion, as follows: "A System of Cost Keeping," by Myron S. Falk, Assoc. M. Am. Soc. C. E.; and "The Design of Elevated Tanks and Stand-Pipes," by C. W. Birch-Nord, Assoc. M. Am. Soc. C. E.

These papers were printed in *Proceedings* for March, 1909.

Wednesday, May 19th, 1909.—8.30 P. M.—At this meeting two papers will be presented for discussion as follows: "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," by C. E. Grunsky, M. Am. Soc. C. E.; and "Some Extensive Railroad Surveys, and Their Cost per Mile," by W. S. McFetridge, M. Am. Soc. C. E.

Mr. Grunsky's paper was printed in *Proceedings* for March, 1909, and that by Mr. McFetridge appears in this number of *Proceedings*.

June 2d, 1909.—8.30 P. M.—Two papers will be presented for discussion as follows: "Tests of Built-Up Steel and Wrought-Iron Compression Pieces," by Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.; and "Caisson Disease and Its Prevention," by Henry Japp, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

ANNUAL CONVENTION

The Forty-first Annual Convention of the Society will be held at the Mount Washington Hotel, Bretton Woods, N. H., from **July 6th** to July 9th, 1909, inclusive.

The general arrangements for the Convention are in the hands of the following Committees:

COMMITTEE OF THE BOARD OF DIRECTION

F. W. Hodgdon,

G. W. Tillson, Chas. Warren Hunt.

LOCAL COMMITTEE

J. W. Ellis,

A. W. Dean, J. F. Stevens,
H. W. Hayes, S. E. Tinkham,
George A. Kimball, H. D. Woods,

CONSERVATION OF NATURAL RESOURCES

A meeting of engineers was held in the Engineering Societies Building, 29 West 39th Street, New York City, at 8 P. M. on Wednesday, March 24th, 1909.

This meeting was called jointly by the four National Engineering Societies, each having selected one of its members to deliver an address on the Conservation of Natural Resources.

Dr. James Douglas, Past-President of the American Institute of Mining Engineers, presided. A telegram from President William H. Taft was read, and Dr. Douglas addressed the meeting.

Addresses were then delivered as follows:

"The Conservation of Water,"
John R. Freeman, M. Am. Soc. C. E.;

"The Conservation of Natural Resources by Legislation,"
R. W. Raymond, M. Am. Inst. M. E.;

"The Waste of Our Natural Resources by Fire,"
Charles Whiting Baker, M. Am. Soc. M. E., and

"Electricity and the Conservation of Energy,"
Lewis B. Stillwell, M. Am. Inst. E. E.

These addresses will not be published in *Proceedings,* but are being printed and will be mailed, in separate pamphlet form, to all members of the Society.

THE JOHN FRITZ MEDAL

The John Fritz Medal was established by the professional associates and friends of John Fritz, of Bethlehem, Pennsylvania, U. S. A., on August 21st, 1902, his eightieth birthday, to perpetuate the memory of his achievements in industrial progress. The rules governing the award provide that the medal shall be given "for notable scientific or industrial achievement, and that there shall be no restriction on account of nationality or sex." Awards may be made annually by a Board of sixteen, appointed or chosen in equal numbers from the membership of the four National Engineering Societies, the American Society of Civil Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers.

The awards made to date have been as follows:

First award, January 20th, 1905, to Lord Kelvin for his work in cable telegraphy and other scientific attainments.

Second award, January 19th, 1906, to George Westinghouse for the invention and development of the air brake.

Third award, January 18th, 1907, to Alexander Graham Bell for the invention and introduction of the telephone.

Fourth award, January 17th, 1908, to Thomas Alva Edison for the invention of the duplex and quadruplex telegraph; the phonograph; the development of a commercially practical incandescent lamp; the development of a complete system of electric lighting, including dynamos, regulating devices, underground system protective devices and meters.

Fifth award, January 16th, 1909, to Charles T. Porter for his work in advancing the knowledge of steam engineering and in improvements in engine construction.

Board by which the last award was made:

E. G. Spilsbury, Chairman; Chas. Warren Hunt, Secretary; C. C. Schneider, Frederic P. Stearns, Geo. H. Benzenberg, James Douglas, Charles Kirchhoff, E. E. Olcott, John E. Sweet, Henry R. Towne, Ambrose Swasey, F. R. Hutton, John W. Lieb, Jr., S. S. Wheeler, Samuel Sheldon, H. G. Stott.

The medal was presented to Charles Talbot Porter on Tuesday evening, April 13th, 1909, in the Engineering Societies Building, New York City.

The presentation was made by E. Gybbon Spilsbury, Past-President, Am. Inst. E. E.; M. Am. Soc. C. E.; Chairman of the Board of Award; and the following addresses were delivered:

"The Debt of Modern Industrial Civilization to the Steam Engine as a Source of Power."

W. F. M. Goss, M. Am. Soc. M. E.; Assoc. Am. Inst. E. E.; Dean of the College of Engineering of the University of Illinois.

"The Debt of the Modern Steam Engine to Charles T. Porter."

F. R. Hutton, Hon. Sec., Am. Soc. M. E.

"The Debt of the Era of Steel to the High-Speed Engine."

Robert W. Hunt, M. Am. Soc. C. E.; Past-President, Am. Soc. M. E.; Past-President, Am. Inst. M. E.

"The Debt of the Era of Electricity to the High-Speed Steam Engine."

Frank J. Sprague, Past-President, Am. Inst. E. E.; M. Am. Soc. C. E.

PAPERS AND DISCUSSIONS

The first volume of *Transactions* for 1909 (Vol. LXII) has been issued. There will be three additional volumes issued during the year.

It is hoped that members and others who take part in the discussion of the papers presented will revise their remarks promptly, and that all written communications from those who cannot attend the meetings will be sent in at the earliest possible date after the issue of a paper in *Proceedings*. The issue of volumes of *Transactions* is

dependent on the closing of discussions, and the co-operation of the membership will now be more necessary in this matter than heretofore, because four volumes are to be issued during the year instead of two, and, to accomplish this, a definite date of issue for each has been established. It is expected that the second volume of 1909 will be issued on June 30th, and the third and fourth on September 30th and December 31st, respectively.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers, which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and, on these, oral discussion, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions, only, will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms and at all Meetings:

American Institute of Mining Engineers, 29 West Thirty-ninth Street, New York City.

Associação dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 197 Montague Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 877 Dorchester Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, 718 Caxton Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Colorado Association of Members, Am. Soc. C. E., 235 Equitable Building, Denver, Colo.

Engineers' and Architects' Club of Louisville, Ky., 303 Norton Building, Fourth and Jefferson Streets, Louisville, Ky.

Engineers' Club of Baltimore, Baltimore, Md.

Engineers' Club of Central Pennsylvania, Corner Second and Walnut Streets, Harrisburg, Pa.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Society of Western Pennsylvania, 803 Fulton Building, Pittsburg, Pa.

Institute of Marine Engineers, 58 Romford Road, Stratford, London, E., England.

Institution of Engineers of the River Plate, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, 604 Tulane-Newcomb Building, New Orleans, La.

Memphis Engineering Society, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Montana.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oesterreichischer Ingenieur- und Architekten-Verein, Eschenbachgasse 9, Vienna, Austria.

Pacific Northwest Society of Engineers, 617-618 Pioneer Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sachsischer Ingenieur- und Architekten-Verein, Dresden, Germany.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Societe des Ingenieurs Civils de France, 19 Rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., England.

Svenska Teknologföreningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members, who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

In reference to this work the Appendix* to the Annual Report of the Board of Direction for the year ending December 31st, 1906, contains a summary of all searches made to that date.

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* *Proceedings*, Vol. XXXIII, p 20 (January, 1907).

ACCESSIONS TO THE LIBRARY

(From March 10th to April 12th, 1909.)

DONATIONS.*

SURVEYOR'S HAND BOOK.

By T. U. Taylor, M. Am. Soc. C. E. Roan, $6\frac{3}{4}$ x $4\frac{1}{2}$ in., illus., 13 + 310 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $2.00 net.

The purpose of the author, as stated in his preface, has been to prepare, for the use of the surveyor in the field, a book of convenient size and scope, and one containing all the essentials for ordinary surveying. The traverse table has been omitted, as, in the author's opinion, small traverse tables are of no avail in an exact survey and the large ones are books in themselves. Several of the chapters have short bibliographies at the end. The Contents are: Chain Surveying; Compass Surveying; Transit Surveying; Calculation of Areas; Division of Land; Leveling; Topographic Survey; Railroad Surveying; Earthwork; City Surveying; Plotting and Lettering; Government Surveying; Trigonometric Formulas; Tables; Index.

ENGINEERING WORK IN TOWNS AND CITIES.

By Ernest McCullough. Second Edition. Cloth, 8 x $5\frac{1}{2}$ in., illus., 8 + 502 pp. Chicago and New York, The Myron C. Clark Publishing Co., 1908. $3.00 net.

This work is written for two classes of town officials, those with no technical education, as councilmen, etc., and those having more or less technical training, as town or city engineers. In the first edition each subject was treated in two chapters, one from each standpoint, but in this edition the two treatments have been combined so that every subject is complete for all classes of readers in the chapter devoted to it. Where errors were found, the preface states, they have been corrected in this edition, some new matter has been added, and the chapter on Reinforced Concrete entirely rewritten. The author states also that much of the information is the accumulation of many years' experience, and that he believes some of it to be here printed for the first time, as it has been picked up in the field, over the drawing board, at engineers' conventions and in conversation with experienced and practical men. The Contents are: The City Engineer and His Duties; Roads and Streets; Walks, Curbs and Gutters; Street Pavements; Sanitation; Drainage and Sewerage; Water Supply; Concrete; Contracts and Specifications; Miscellaneous Data; Office Systems; City Engineer's Records; Field Work; Engineering Data; Index.

THE DESIGN AND EQUIPMENT OF SMALL CHEMICAL LABORATORIES.

By Richard K. Meade. Cloth, $9\frac{1}{4}$ x 6 in., illus., 136 pp. Chicago, The Chemical Engineer Publishing Co., 1908. $2.00 net.

The author states that while this book is intended primarily as suggestions to young and inexperienced chemists, he hopes experienced chemists will find in it some points of use and help. The firms making the various forms of apparatus described are mentioned by name for the convenience of the reader. The Contents are: General Features; Hoods; Sinks and Water Supply; Desks; Table and Apparatus for Rapid Filtrations, etc.; Ignition Table and Apparatus for Ignition; Table and Apparatus for Titrations; Balance Support, Balance, and Accessories; Heating Appliances; Preparation of Distilled Water; Apparatus for Electro-chemical Analysis; Sampling Appliances; Assay Furnaces and Accessories; Miscellaneous Laboratory Equipment; Index.

THE ENGINEERING INDEX ANNUAL FOR 1908.

Compiled from the Engineering Index Published Monthly in *The Engineering Magazine* during 1908. Cloth, $9\frac{1}{2}$ x $6\frac{1}{2}$ in., 14 + 437 pp. New York and London, The Engineering Magazine, 1909. $2.00.

This is the seventh volume of the Engineering Index issued since the work was first undertaken by the late J. B. Johnson, M. Am. Soc. C. E., and the third since

*Unless otherwise specified, books in this list have been donated by the publisher.

the index became an annual. Taken together these volumes form a continuous index to engineering and technical literature from the beginning of 1884. The arrangement of the index is still the same as that published monthly in *The Engineering Magazine*. The articles are first grouped under the great divisions of engineering practice—Civil, Mechanical, Electrical, Mining, etc.—and under these again sub-grouped according to the recognized special divisions of each field, the final arrangement under these being strictly alphabetical. The 1908 Annual includes about two hundred and fifty publications, about three-fourths of which are printed in English, the others being in German, French, Spanish, Italian, and Dutch. Following each entry is a brief descriptive note defining the scope and purport of the article. Serials are indexed under the first installment except in the case of short articles, which are indexed entire. The Annual is carried through 1908, but a continuation of it is issued monthly in *The Engineering Magazine.*

THEORY OF SOLID AND BRACED ELASTIC ARCHES.

By William Cain, M. Am. Soc. C. E. Second Edition, Revised and Enlarged. Boards, 6 x 3¾ in., illus., 8 + 100 pp. New York, D. Van Nostrand Company, 1909. 50 cents. (Donated by the Author.)

This work has been, the author states, almost entirely rewritten, and treats in great detail of elastic arches of variable cross-section, particularly concrete and reinforced concrete arches, the aim being, not only to present the theory in a simple, convincing manner, but to furnish the computer with several examples worked out in full, so that he shall find no difficulty in applying the theory rapidly and with certainty, to similar arches. The Chapter headings are: Equilibrium Polygons, Elastic Theory; Concrete Arch, Both Graphical and Analytical Solutions; Three-Centered Arch, Algebraic Solution in Full; Temperature and Allied Stresses; Reinforced Concrete Arch, Method of Single Loads; Arches with Two or Three Hinges, Braced Arches.

UNIVERSITY OF LONDON, CHADWICK LECTURES, SESSION 1907-08.

Containing Lectures on the Engineering Aspect of Recent Advances in Connection with Sewering. By W. D. Scott-Moncrieff. Paper, 8½ x 5½ in., 79 pp. London, St. Bride's Press, Ltd., 1909. Two shillings net.

This series of four lectures was delivered at the University of London under the endowment derived from the Chadwick Trust, which requires that the lectures shall have special reference to recent advances in hygiene and municipal engineering. Lecture I is historical and introductory, and takes up the early unsanitary condition of England, and summarizes the progress made, reports of committees, etc.; Lecture II deals with the functions of the physician, chemist, bacteriologist, and engineer in sanitary progress, and the special work of the engineer in connection with the sewering of houses and streets; Lecture III is on sewage disposal; Lecture IV discusses what the engineer, engaged in sewage disposal work, should know with regard to chemistry, bacteriology, etc.

THE RAILROAD SIGNAL DICTIONARY

An Illustrated Vocabulary of Terms Which Designate American Railroad Signals, Their Parts, Attachments, and Details of Construction, With Descriptions of Methods of Operation and Some Illustrations of British Signals and Practice. Comp. for the Railway Signal Association by Braman B. Adams and Rodney Hitt, Jun. Am. Soc. C. E. Morocco, 12 x 8½ in., illus., 10 + 36 + 472 pp. New York and Chicago, Railroad Age-Gazette; London, Railway Gazette, 1908. $6.00.

This work consists of thirty-two pages of dictionary text and more than four hundred pages of illustrations interspersed with a small amount of text. At the end of each dictionary article there are references to the figures in the album part of the volume. The illustrations are grouped in five parts: Signal Indications; Block Signals; Highway Crossing Signals; Interlocking; Block and Interlocking Accessories. There are classified and alphabetical indexes given at the beginning of the album portion of the volume, making the collection of engravings accessible to the user.

Gifts have also been received from the following:

Am. Gas Inst. 1 bound vol.
Argentine Republic-Ministerio de Obras Publicas. 1 pam.
Arizona-Agri. Exper. Station. 5 pam.
Arlington, Mass.-Town Clerk. 6 vol.
Arnold, Bion J. 3 pam.
Australia-Commonwealth Statistician. 3 pam.
Belzner, Theodore. 2 pam.
Berlin Kgl. Technische Hochschule. 1 pam.
Boston, Mass.-Health Dept. 1 bound vol., 1 pam.
Brookline, Mass.-Water Dept. 1 pam.
Bureau of Explosives. 3 pam.
California-State Forester. 2 pam.
California-State Harbor Commrs. 3 pam.
Canada-Dept. of Rys. and Canals. 1 vol.
Canadian Min. Inst. 2 pam.
Canadian Soc. of Civ. Engrs. 2 pam.
Chicago, Ill.-Dept. of Smoke Prevention. 3 pam.
Chicago, Indiana & Southern R. R. Co. 1 pam.
Civ. Engrs.' Soc. of St. Paul, Minn. 1 pam.
Cleveland Eng. Soc. 1 pam.
Colorado-State Engr. 2 bound vol.
Columbus, Ohio-Chf. Engr. of Dept. of Public Service. 1 pam.
Detroit, Mich.-Board of Water Commrs. 2 pam.
Eastern Bengal and Assam-Public Works Dept. 1 pam.
Fitchburg, Mass.-City Engr. 1 pam.
Glasgow & South-Western Ry. Co. 1 pam.
Graduates Club of New York City. 1 bound vol.
Great Central Ry. Co. 1 pam.
Great North of Scotland Ry. Co. 2 pam.
Green Bay & Western R. R. Co. 1 pam.
Greene, G. S. 19 bound vol., 24 pam.
Highland Ry. Co. 1 pam.
John Crerar Library. 1 pam.
Johnstown, N. Y.-Board of Water Commrs. 1 pam.
Lake Erie & Western R. R. Co. 1 pam.
Lake Shore & Mich. Southern Ry. Co. 1 pam.
Leominster, Mass.-Water Board. 1 pam.
Maine-Commr. of Highways. 1 bound vol.
Merchants' Assoc. of New York. 2 pam.
Mich. Central R. R. Co. 1 pam.
Michigan Technic. 1 pam.
National Assoc. of Cotton Mfrs. 1 bound vol.
Nebraska-State Engr. 7 bound vol.
New Jersey-Commr. of Public Roads. 1 bound vol.
New Jersey-State Board of Health. 1 pam.
New Jersey San. Assoc. 1 pam.
New York City-Board of Estimate and Apportionment. 1 pam.
New York City-Mayor. 1 vol.
New York State-Public Service Comm., First Dist. 1 pam.
New York State-Public Service Comm., Second Dist. 2 bound vol., 1 pam., 1 map.
New York-State Engr. and Surv. 1 map.
New York Central & Hudson R. R. Co. 1 pam.
New York City Record. 1 bound vol.
Newark, N. J.-Board of Street and Water Commrs. 2 pam.
Newhall, Henry. 37 pam.
North, Edward P. 2 bound vol.
North British Ry. Co. 1 pam.
Nova Scotia Min. Soc. 1 vol.
Oberlin College Library. 1 pam.
Ohio-State Board of Health. 1 bound vol.
Ohio State Univ. 1 vol., 1 pam.
Oesterreichischer Ingenieur- und Architekten-Verein. 4 pam.
Pennsylvania R. R. Co. 1 pam.
Philippine Islands-Bureau of Forestry. 3 pam.
Philippine Islands-Bureau of Science. 2 pam.
Punjab, India-Irrig. Dept. 1 bound vol.
Randle, George N. 6 pam.
Rensselaer Polytechnic Inst. 1 pam.
Rhode Island-R. R. Commr. 1 bound vol.
Royal Philosophical Soc. of Glasgow. 1 vol.
St. Louis, Mo., Merchants' Exchange. 1 pam.
St. Paul, Minn.-Board of Water Commrs. 1 pam.
St. Paul, Minn.-City Comptroller. 1 bound vol.
Saskatchewan, Canada-Dept. of Public Works. 1 vol.
Snare & Triest Co. 1 bound vol.
Société Belge des Ingénieurs et des Industriels. 4 pam.
Soc. of Engrs. 1 pam.
Soc. of Naval Archts. and Marine Engrs. 1 pam.
Texas-Tax Commr. 1 pam.
Texas & Pacific Ry. Co. 1 pam.
Transvaal-Mines Dept. 1 vol.
U. S.-Bureau of Insular Affairs. 1 vol.
U. S.-Bureau of Labor. 1 bound vol.
U. S.-Bureau of Standards. 4 pam.
U. S.-Chief of Engrs. 2 vol.
U. S.-Coast and Geodetic Survey. 1 vol., 1 pam.
U. S.-Copyright Office. 1 pam.
U. S.-Dept. of Agriculture. 1 pam.
U. S.-Geol. Survey. 2 bound vol., 4 vol., 5 pam.
U. S.-Geol. Survey-Technologic Branch. 1 pam.
U. S.-Interstate Commerce Comm. 3 bound vol.
U. S.-Isthmian Canal Comm. 1 pam.
U. S.-Office of Library and Naval War Records. 1 pam.
U. S. Steel Corporation. 1 pam.
Univ. of Cal.-Univ. Press. 9 vol.
Univ. of Illinois-Eng. Exper. Station. 1 bound vol.
Univ. of Vermont and State Agrl. Coll. 1 pam.
Verein Deutscher Eisenhüttenleute. 1 vol.
Waltham, Mass.-Water Dept. 1 pam.
Wauters, Carlos. 4 pam.
Wellington, New Zealand-Harbour Board. 1 pam.
Western Australia-Geol. Survey. 2 pam.
Wilgus, William J. 1 bound vol.
Wis. Valley Improvement Co. 1 pam.

BY PURCHASE

Electrical Illuminating Engineering. By W. E. Barrows, Jr. McGraw Publishing Co., New York, 1908.

Design and Construction of Ships. By John Harvard Biles. Vol. 1, Calculations and Strength. Charles Griffin & Co., Ltd., London, 1908.

The Railway Locomotive. What It Is and Why It Is What It Is. By Vaughan Pendred. Archibald Constable & Co., Ltd., London, 1908.

Atlas of Canada. Prepared under the Direction of James White, Geographer. Department of the Interior, Canada, 1906.

Public Health. Papers and Reports Presented at the Thirty-fifth Annual Meeting of the American Public Health Association, Atlantic City, New Jersey, Sept. 30th, Oct. 1st, 2d, 3d, 4th, 1907. Vol. XXXIII, Parts I and II. Columbus, Ohio, Fred. J. Heer, 1908.

Handbuch über Triebwagen für Eisenbahnen. Von C. Guillery. R. Oldenbourg, München und Berlin, 1908.

Über den Querschnitt der Staumauern. Von Ferd. Platzmann. Fortschritte der Ingenieurwissenschaften. Wilhelm Engelmann, Leipzig, 1908.

Guide to Sanitary Inspections. By W. P. Gerhard. Ed. 4, rev. and enl. John Wiley & Sons, New York; Chapman & Hall, London, 1909.

High-Tension Underground Electric Cables: A Practical Treatise for Engineers. By Henry Floy. Electrical Publishing Co., New York, 1909.

SUMMARY OF ACCESSIONS

(From March 9th to April 12th, 1909)

Donations (including 56 duplicates)	272
By purchase	10
Total	282

MEMBERSHIP

ADDITIONS

(March 10th to April 13th, 1909)

MEMBERS		Date of Membership
ALLARD, THOMAS THROP. 101 Obispo St., Havana, Cuba....		Mar. 2, 1909
BASCOM, HARRY FRANKLIN. Engr. (Bascom & Sieger), 603 Allentown Bank Bldg., Allentown, Pa.	Jun. Assoc. M. M.	Dec. 5, 1899 Sept. 4, 1901 Mar. 2, 1909
BRAZER, GEORGE HERBERT. (J. R. Worcester & Co., Cons. Engrs.), 79 Milk St., Boston, Mass.	Assoc. M. M.	Nov. 1, 1905 April 6, 1909
CRELLIN, EDWARD WEBSTER. Pres., Des Moines Bridge & Iron Works, Des Moines, Iowa.		Mar. 2, 1909
FALK, MYRON SAMUEL. Cons. Engr., 30 Church St. (Res., 44 West 53d St.), New York City.	Jun. Assoc. M. M.	Feb. 4, 1902 Oct. 4, 1905 April 6, 1909
HOUSTON, GAVIN NELSON.. Deputy State Engr., State House, Denver, Colo.	Jun. Assoc. M. M.	May 4, 1897 Nov. 7, 1900 Feb. 2, 1909
KELSEY, LOUIS CURTIS. City Engr., Salt Lake City, Utah..		Mar. 2, 1909
LACKEY, OSCAR FRANCIS. Harbor Engr., Baltimore, Md.	Assoc. M. M.	May 4, 1904 April 6, 1909
MACHEN, HENRY BENNETT. Asst. Engr., Dept. of Water Supply (Res., 315 West 97th St.), New York City.	Jun. Assoc. M. M.	May 2, 1899 Feb. 1, 1905 April 6, 1909
McMURTRY, GUY. Yuba City, Cal.		Mar. 2, 1909
MORRISON, THOMAS JOHN. 112 Cutler Bldg., Rochester, N. Y.		April 6, 1909
NELSON, JAMES AUGUSTUS. 800 Westinghouse Bldg., Pittsburg, Pa.		Mar. 2, 1909
OESTREICH, HENRY LEWIS. Asst. Engr., Public Service Comm., 154 Nassau St. (Res., 2092 Bathgate Ave.), New York City....	Jun. Assoc. M. M.	Oct. 4, 1892 Dec. 6, 1899 April 6, 1909
PALMER, JOHN GEARY. Barge Canal Office, Montezuma, N. Y.		Oct. 7, 1908
RAY, FREDERICK GEORGE. 33 Campau Bldg., Detroit, Mich..		April 6, 1909
SERBER, DAVID CHARLES. Structural Engr., Henry Steers, Inc., 17 Battery Pl., New York City.		April 6, 1909
SMITH, GILMAN WALTER. Div. Erecting Mgr., Am. Bridge Co. of N. Y., Commercial National Bank Bldg., Chicago, Ill.		Mar. 2, 1909
STEVENS, HOWARD EVELETH. Bridge Engr., N. P. Ry. Co., St. Paul, Minn.		April 6, 1909
WEBSTER, ALBERT LOWRY. Cons. Engr., 82 Wall St., New York City.	Jun. Assoc. M. M.	Sept. 6, 1882 June 3, 1891 April 6, 1909

MEMBERS (*Continued*)

		Date of Membership
WHITE, WILLIAM MACINTIRE. Engr., United States Steel Products Export Co., 30 Church St., Room 1320, New York City..	Assoc. M. M.	Jan. 2, 1901 Mar. 2, 1909
WITT, CARLTON CARPENTER. Engr., South Dakota R. R. Comm., Sioux Falls, S. Dak		April 6, 1909

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ACKERMAN, ALEXANDER SEYMOUR. Asst. Engr., Cape Cod Canal Co., Sandwich, Mass....	Jun. Assoc. M.	April 30, 1907 April 6, 1909
BLAIR, MCCREA PARKER. City Engr., City Hall, St. Boniface, Man., Canada		Feb. 2, 1909
BLAYLOCK, JOHN CHARLES. 1487 Wellington St., Chicago, Ill.		Mar. 2, 1909
BRENEMAN, PAUL BRUCE. Box 294, State College, Pa......		Mar. 2, 1909
BUTZ, GEORGE WISHART. Civ. and Cons. Engr., Schuylkill Haven, Pa		April 6, 1909
CARRICK, ROBERT EDWARD. Engr., The Gen. Fireproofing Co., 257 East 133d St., New York City	Jun. Assoc. M.	April 30, 1907 April 6, 1909
DICKE, EDWARD CHRISTIAN. 1909 Hebert St., St. Louis, Mo	Jun. Assoc. M.	May 3, 1904 April 6, 1909
FEDERLEIN, WALTER GOTTLIEB. 2644 Marion Ave., New York City	Jun. Assoc. M.	Jan. 2, 1906 Feb. 2, 1909
FORREST, CHARLES NEEDHAM. Maurer, N. J		April 6, 1909
HAMILL, ALEXANDER SYLVESTER. 426 Montgomery St., Jersey City, N. J	Jun. Assoc. M.	Dec. 28, 1900 April 6, 1909
IJAMS, JESSE WARREN. Asst. Engr., N. Y. C. & H. R. R. R. Co., Room 32, Union Station, Albany, N. Y		April 6, 1909
JUDD, FRANK RHYMAL. 6543 Minerva Ave., Chicago, Ill....		Mar. 2, 1909
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MACARTNEY, MORTON. P. O. Box 2241, Spokane, Wash.....		Mar. 2, 1909
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MILLER, HIRAM. Care, Bolivia Ry. Co., La Paz (via Panama), Bolivia		Jan. 5, 1909
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MURPHY, JOHN JOSEPH. 70 Park Hill Ave., Yonkers, N. Y..		Mar. 2, 1909
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PECK, MYRON HALL. Prof. of Civ. Eng., Imperial Pei Yang Univ., Hsi Ku, Tientsin, China		Jan. 5, 1909
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SMITH, THOMAS ARTHUR. P. O. Box 807, Jacksonville, Fla.		April 6, 1909
ULRICH, EDMUND BOYD. 303 Baer Bldg., Reading, Pa.		April 6, 1909
WICKERSHAM, JOHN HOUGH. Lancaster, Pa.	Jun. Assoc. M.	May 6, 1902 April 6, 1909
YOUNG, LEWIS MAXWELL. 119 Trenton St., East Boston, Mass.		April 6, 1909

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JUNIORS

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DEATHS

BREEN, HOWARD. Elected Member, April 4th, 1888; died February 14th, 1909.

DEANS, CHARLES HERBERT. Elected Junior, December 3d, 1890; Associate Member, May 6th, 1896; died March 7th, 1909.

SCHAUB, JULIUS WILLIAM. Elected Junior, November 5th, 1884; Member, October 6th, 1886; died March 30th, 1909.

WILLARD, JAMES EAGER. Elected Member, May 1st, 1889; died March 8th, 1909.

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(March 9th to April 12th, 1909)

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LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(1) *Journal,* Assoc. Eng. Soc., 31 Milk St., Boston, Mass., 30c.
(2) *Proceedings,* Engrs. Club of Phila., 1317 Spruce St., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Monadnock Blk., Chicago, Ill.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly,* Columbia Univ., New York City, 50c.
(7) *Technology Quarterly,* Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator,* Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine,* New York City, 25c.
(10) *Cassier's Magazine,* New York City, 25c.
(11) *Engineering* (London). W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c
(13) *Engineering News,* New York City, 15c.
(14) *The Engineering Record,* New York City, 12c.
(15) *Railroad Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway and Engineering Review,* Chicago, Ill., 10c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 10c.
(21) *Railway Engineer,* London, England, 25c.
(22) *Iron and Coal Trades Review,* London, England, 25c.
(23) *Bulletin,* American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal,* New York City, 10c.
(25) *American Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(41) *Modern Machinery,* Chicago, Ill., 10c.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Mines and Minerals,* Scranton, Pa., 20c.
(46) *Scientific American,* New York City, 8c.
(47) *Mechanical Engineer,* Manchester, England.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Verein, Vienna, Austria.
(54) *Transactions,* Am. Soc. C. E., New York City, $5.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $5.

(57) *Colliery Guardian,* London, England.
(58) *Proceedings,* Eng. Soc. W. Pa., 803 Fulton Bldg., Pittsburg, Pa., 50c.
(59) *Transactions,* Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Industrial World,* 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 20c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting,* London, England, 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(68) *Mining Journal,* London, England.
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(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 10c.
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(78) *Beton und Eisen,* Vienna, Austria.
(79) *Forscherarbeiten,* Vienna, Austria.
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(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(82) *Dinglers Polytechnisches Journal,* Berlin, Germany.
(83) *Progressive Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. and M. of W. Assoc., Chicago, Ill.
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(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
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(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(94) *The Boiler Maker,* New York City, 10c.
(95) *International Marine Engineering,* New York City, 20c.

LIST OF ARTICLES.

Bridge.

The Sinking of the Piers for the Grand Trunk Pacific Bridge at Fort William, Ontario, Canada.* H. L. Wiley, Jun. Am. Soc. C. E. (54) Vol. 62.
Lift Bridges; a Discussion.* (58) Feb.
Secondary Stresses in Framed Structures.* E. W. Pittman. (58) Feb.
The Largest Concrete Arch ever Built.* (67) Mar.
The Widening of Blackfriars Bridge.* (11) Mar. 5.
The Stress Sheets and Some Construction Details of a 280-ft. Span Rubble Concrete Arch Bridge at Cleveland, O.* (86) Mar. 10.
Handling Complete Bridge Spans.* (15) Mar. 12.
Comparative Construction and Maintenance Costs of Reinforced Concrete and Steel Bridges. E. P. Goodrich. (Paper read before the National Cement Users Assoc.) (86) Mar. 17.
Bascule Bridges over the East Chicago Canal.* (13) Mar. 18.
Experimental Verification of Arch Formulas.* Malverd A. Howe. (15) Mar. 26.
Concrete Bridge Specifications. T. H. MacDonald. (Paper read before the Iowa Cement Users Assoc.) (60) Apr.
The Atlantic Avenue Bridge Extension, Boston.* (14) Apr. 3.
Erection of Delaware River Bridge at Portland, Pennsylvania.* (14) Apr. 3.
The Erection of the Susquehanna River Bridge of the Baltimore & Ohio Railroad.* (14) Apr. 3.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.

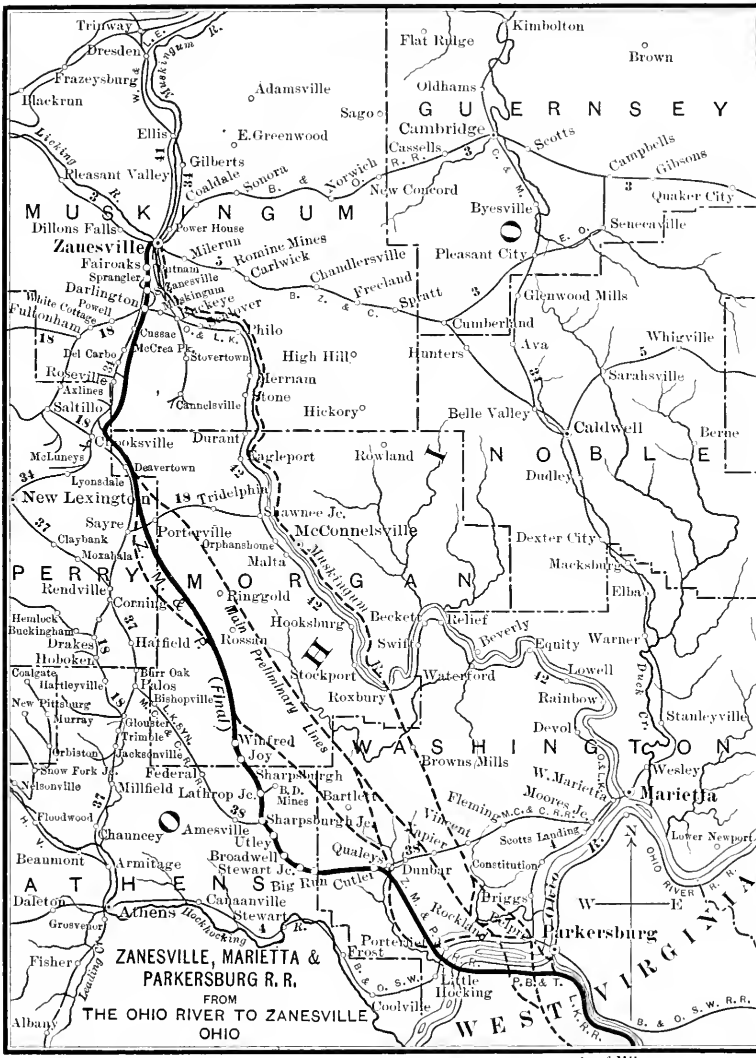
By W. S. McFetridge, M. Am. Soc. C. E.

To be Presented May 19th, 1909.

The following paper is not a theoretical discussion of railroad location. It is intended to give a general description of some extensive railroad surveys, a brief outline of methods and results, and the cost per mile. The writer does not remember having seen a similar statement covering so many miles, viz., 1 400 miles of preliminary lines and 600 miles of location; and therefore trusts the paper may prove of value. The surveys were completed some time ago, at which time the tables of mileage and cost were prepared, but not heretofore published.

Introductory.—Early in 1902 the Little Kanawha Syndicate began surveys for the extension of its lines, eastward from Palestine, W. Va., to Belington, and westward from Parkersburg, W. Va., to Zanesville, Ohio. About one and one-half years later it also took up the location and construction of a line running northward from Belington to the Pennsylvania-West Virginia State line.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.



3. Baltimore & Ohio R. R.
4. " " " S.W. R.R.
5. Bellaire, Zanesville & Cincinnati Ry.
18. Columbus, Sandusky & Hocking Ry.
34. Cincinnati & Muskingum Valley Ry.
37. Toledo & Ohio Central Ry.
38. Marietta, Columbus & Cleveland R. R.
41. Wheeling & Lake Erie Ry.
42. Ohio & Little Kanawha R. R.

Fig. 1.

The surveys were conducted under the following charters: Zanesville, Marietta and Parkersburg Railroad, in Ohio; Parkersburg Bridge and Terminal Railroad, from the Ohio-West Virginia State line to Parkersburg (this division included a bridge over the Ohio River a few miles below Parkersburg); Little Kanawha Railroad, from Parkersburg to Burnsville; Burnsville and Eastern Railroad, from Burnsville to Belington; Buckhannon and Northern Railroad from Belington to the Pennsylvania-West Virginia State line; in all, some 328 miles of main-line location, exclusive of branch lines.

Fig. 1 (Ohio) and Plate X (West Virginia) show the country traversed and the main survey lines, many of the short lines not being shown. The lines shown give the general layout. The termini, as usual, were fixed; physical conditions also fixed the Little Kanawha River as the only outlet to the Ohio. These points decided in a general way the proposed route. Owing to local conditions, it was also believed that the heavier traffic would be west-bound, and therefore that every effort should be made to get as low a ruling grade as possible for this traffic:

The desired results may be briefly stated as follows:

1.—Easiest grades possible, especially against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line;

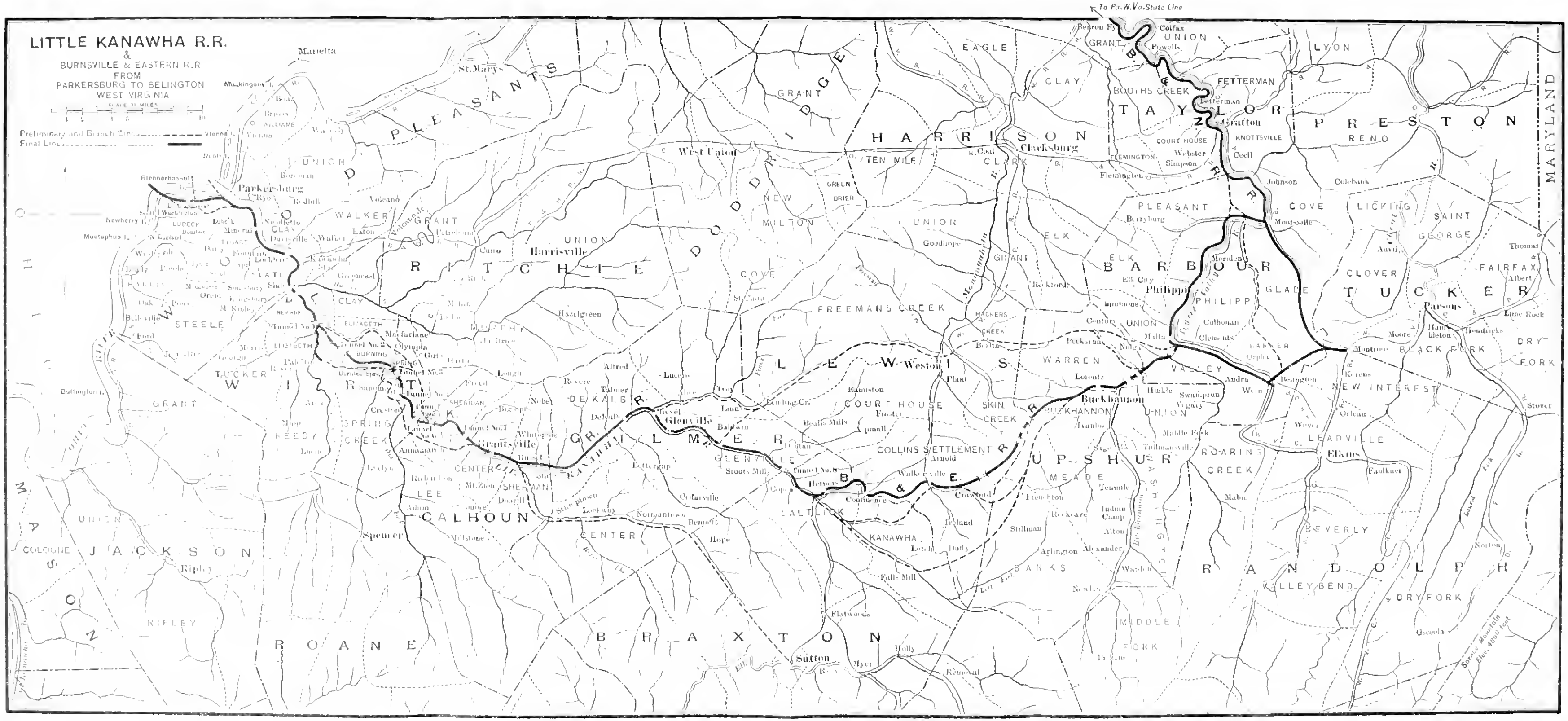
4.—Occupy to as great an extent as possible critical and strategical points, so as to block the route to other lines;

5.—Reach certain definite places previously determined on;

6.—And, taking into consideration the naturally heavy and expensive work on any line, the total cost should compare favorably with the cost of any other line, many miles longer, with heavier grades and curves, but avoiding some of the most expensive work.

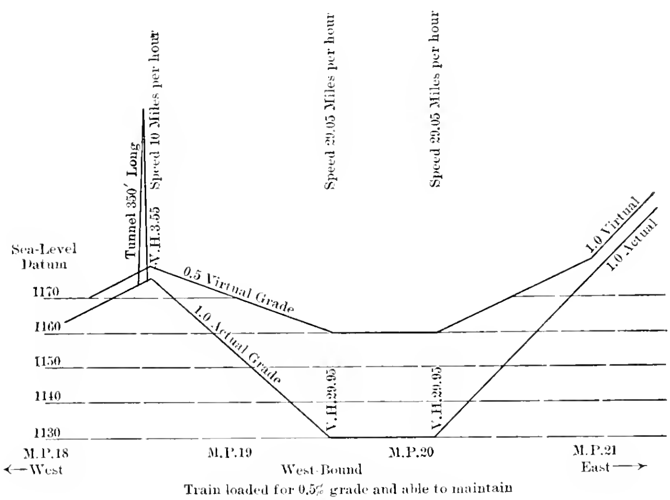
All roads previously built through the adjoining regions have long stretches of 1.5% grades, and curves up to 12 and 14 degrees. The first surveys, therefore, were of a preliminary nature, in order to determine what grades and curves could be secured.

After a number of surveys, locations, and explorations had been made, it was found that the following grades and curves were possible; in Ohio, 0.5% grades, 4° maximum curve; Little Kanawha Division, 0.3% grades, 8° maximum curve; Burnsville and Eastern Division,


LITTLE KANAWHA R.R.
&
BURNSVILLE & EASTERN R.R
FROM
PARKERSBURG TO BELINGTON
WEST VIRGINIA
Preliminary and Branch Lines
Final Lines
To Pa.W.Va.State Line
PLEASANTS
WOOD
RITCHIE
DODDRIDGE
HARRISON
TAYLOR
PRESTON
MARYLAND
BARBOUR
TUCKER
LEWIS
UPSHUR
RANDOLPH
GILMER
CALHOUN
WIRT
JACKSON
ROANE
BRAXTON
MASON
Parkersburg
Clarksburg
Grafton
Philippi
Weston
Buckhannon
Glenville
Grantsville
Spencer
Sutton
Elkins
Parsons
Harrisville
Ripley

1.0% grades against east-bound and 0.5% grades, against west-bound traffic, 8° maximum curves; all grades compensated for curvature at the rate of 0.04′ per degree.

These results were kept in view in continuing the surveys, and were obtained in each case. It was desired to avoid all momentum grades, and only in one case was it found necessary to use them. This case occurred at Mile 20 on the Burnsville and Eastern Division, where such a grade was introduced in order to avoid a long détour combined with some exceptionally heavy work. At some future time this grade can be taken out if desired. In the meantime, for any ordinary reason of operation, a train need never stop there, and thus get stalled; there is no station stop, and a water-station or siding is not feasible, on account of local conditions, so that trains can always "run for the hill." Fig. 2 is a profile at this place.



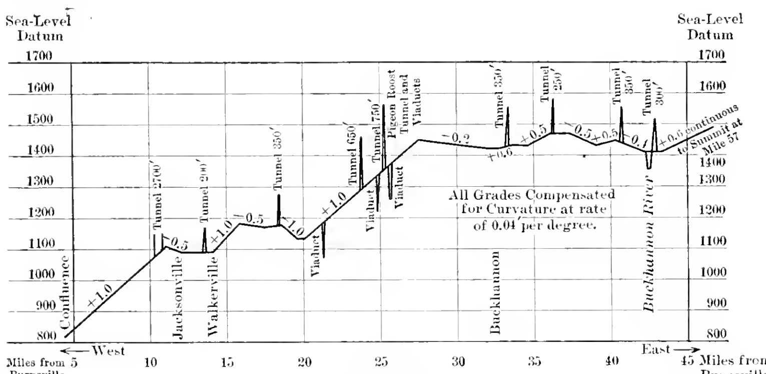
Train loaded for 0.5% grade and able to maintain uniform speed of 10 miles per hour.
BURNSVILLE & EASTERN R.R.
Profile at M.P. 20, showing Virtual Profile for West-Bound Traffic.

FIG. 2.

The grades desired were very easy for parts of the country, and required some rather long continuous grade lines, the longest being on the Burnsville and Eastern Division, where there are 1.0% grades, 7 miles and 7½ miles long, respectively, and a 0.6% grade 14 miles long, all against east-bound traffic. Fig. 3 shows a grade-line profile of this division.

To obtain these grades it was necessary to make a thorough study of the country and a detailed examination of several routes.

The topographical sheets of the United States Geological Survey were found of great value in making a broad, general study of the country. At that time the published sheets covered only a small portion of the lines, but, fortunately, this was the central part of West Virginia, where they were of particular value. It may be pertinent to state that these sheets were usually found quite accurate in regard to main summits, large rivers, and towns, but intermediate details were always more or less inaccurate.



GRADE PROFILE,
BURNSVILLE & EASTERN R.R.,
M.P. 5 TO M.P. 45.

Fig. 3.

A large number of maps of small scale (1 in. to 1 mile or even smaller) were compiled and traced from various State, county, and road maps, on which the several survey lines could be indicated. Such maps are of particular value, and should be the first ones prepared—if possible before any surveys are made.

General Layout of Surveys.—The general direction of the survey, except along the Little Kanawha Division, was almost directly across the general drainage of the country.

In Ohio a direct line between termini was first examined, but was found to be impracticable. This line marked the north and east limits of the country to be examined; certain conditions prevented the location of a line north of it. A systematic examination toward the southwest was then made, and finally a satisfactory line was developed, over which a final location was made. All the streams here lie in

deep, narrow valleys, and are exceptionally crooked. The only feasible way to traverse much of the country was to get up out of the valleys and stay out. Such a method necessitated crossing about 100 ft. above several streams, and running short tunnels between the water-sheds; it also gave the shortest line, the easiest grades, and the lightest curvature. Any attempt to avoid the high crossings or tunnels involved long détours, excessive curvature, and heavier grades. On this division more miles of preliminary lines were run, in proportion to the location, than on any other.

The main problem on the Parkersburg Bridge and Terminal Railroad was the determination of the location for a bridge over the Ohio River. The Government regulations required 90 ft. clear head-room above low water and no piers in the main channel, which necessitated a 700-ft. span. Several feasible locations were surveyed, and complete data were obtained at each place. The location finally adopted is about 5 miles below Parkersburg, and is believed to be the shortest and cheapest railroad bridge crossing the Ohio between Pittsburg and the Mississippi, the 700-ft. span practically clearing the entire channel, the eastern approach being on the only bottom lands in that region which are not subject to overflow, and the western end being against a steep bank rising rapidly from the river. The bridge and viaduct approaches are but little more than half as long as the present bridge at Parkersburg.

The Little Kanawha Division, in general, followed the Little Kanawha River. The hills rise abruptly from the river banks, the slopes often being as steep as $1\frac{1}{2}$ to 1. The river is very crooked, and to follow it gave a long line with much curvature. Much distance could be saved by cutting through the country at various points, but the work was very heavy. The bottom lands are narrow and subject to overflow several times a year, the river sometimes rising 20 ft. in a day. A river line would have to be built along the steep side-hills for most of the way (this being expensive), or else be subject to inundation during high-water seasons. The hills are cut through in all directions by numerous tributary streams. The grade of the streams is from 50 to 80 ft. per mile, their length varying from 1 or 2 to 20 or 30 miles. and all of them terminate against abrupt hills. Many previous surveys of this river had been made, and practically all of them followed the river, abandoning any serious attempt to shorten

the line materially by cutting across country. The first Little Kanawha survey also followed the river, and a complete location was made. This line was run principally for information, and, shortly afterward, was abandoned. The main problem on this division was to locate such a line that its total cost would not greatly exceed the cost of a river line equally well built (though no definite amount was ever fixed for this), that its grades and curves would be as light as by river, and that it would be as short as possible. This problem required the most minute study of the land, a thorough knowledge of its character, the running of many lines, and the comparison of many estimates. On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river for any material distance because the cost would be prohibitive; but when a thorough study began to show the length of line that could be saved by a mile or two of enormous work, the question had a different aspect. The only way to get sufficiently accurate data, on which to base final conclusions, was to make the actual surveys of the different routes.

One item which was always more or less an unknown quantity, but which was the deciding point between two lines, in more than one case, was the treacherous character of some of the side-hills. In a number of places (the same trouble was also encountered on other parts of the lines) the side-hills were very likely to slide. It often happened that quite a large area, and sometimes from 15 to 20 ft. deep, would move bodily down hill for several feet when disturbed by any construction work. To avoid these places as much as possible, and to estimate their probable cost when comparing lines, were difficult problems. On location, a plan sometimes used was as follows: Where the grade was near the foot of a hill, the location was thrown away from the hill, in order to avoid cutting the slopes; this gave a fill where it was not apparently necessary, but it was cheaper in the end. When the grade was high up on the hillside, the opposite was done, for the purpose of getting the roadbed on solid ground. In many cases, however, such places could not be avoided in any way, and it was often found that ground, to all appearances perfectly firm, would slide after construction started. In comparing the various lines, this tendency to slide had to be taken into account, but the estimate of cost was of necessity more or less an approximation. These slides, throughout this country, are frequently a very large item of cost in building, and are usually underestimated.

The line, as finally located, is a combination of river and cross-country line. It is 31 miles shorter than the river, in a total distance of 100 miles. There are eight tunnels, usually short, the longest being 4 000 ft. There are seven river crossings, with main spans from 100 to 300 ft.

The Burnsville and Eastern Division is in the central mountain part of the State. The highest altitude reached is 1 725 ft. above sea level. At the junction with the Little Kanawha line, the altitude is 750 ft. The lines generally run east; the mountains or hills, north and south; the drainage, generally north. There were several intermediate summits to be crossed. It was about as difficult to find the 0.5% grade west as to find the 1.0% grade east. The first line examined was up the head-waters of the Little Kanawha, up Fall Run, and through the head-waters of French Creek. The country was very rough and broken, and supporting ground for grades could not be found. This line also developed the fact that, owing to the rapid rise of the land at the eastern end, it would be necessary to use the maximum grade at once for getting out of the Little Kanawha water-shed, and then follow around the head-waters of the streams to the north, in order to get supporting ground. The line finally located was outside of the Little Kanawha water-shed.

The Buckhannon and Northern Division followed the river for about two-thirds of its length, the other third being cross-country. The general problems of location were very similar to those on the Little Kanawha and Burnsville and Eastern Divisions.

Methods Used.—Field parties were made up as follows:

	Monthly Salary.
Assistant Engineer in charge	$125 to $150
Transitman	85 " 100
Levelman	75
Rodman	65
Head chainman	50
Rear chainman	45
Rear flagman	40
Stakeman	35
Axemen (from two to five)	30
Topographer	65
Tapemen (two)	45
Draftsman (part time)	60

Camp outfits were not used. The parties boarded at houses along the line. This was often a disadvantage, on account of difficulty in getting quarters, especially for a full corps; but, on the other hand, the party could frequently make its headquarters at some town and drive to and from the work, so that probably this method served just as well as furnishing camp outfits.

Each party was given from 40 to 60 miles of line to cover, depending on local conditions.

The following abstracts from the "General Instructions" give an idea of the work required to be done by each party:

"The Assistant Engineer will receive instructions as to when and where preliminary lines will be run, grades proposed to be used, what the line is intended to develop, and the general information necessary in regard to the same. He will then be responsible for the amount and character of the work done by his party, and for the proper development of the country over which he works.

"Topography will be taken on lines as instructed."

(Topography was taken on practically all lines except in Ohio.)

"After preliminaries are run, the Assistant Engineer will be instructed over which line to locate.

"All location will be first projected on topography sheets, etc.

"No location will be assumed as final until examined and revised, if necessary, by the Assistant Chief Engineer, and also approved by the Chief Engineer."

The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence. On location and in particularly difficult country he was to be with the party almost constantly.

After the first route to be examined had been chosen, a preliminary line was run through; then the alternate routes were run, all surveys being tied together; and finally the lines required for a thorough development of all possible routes were run. Although the first preliminary line might give the grades and curves desired, and possibly be the line over which the final location was made, it was not finally determined on until the subject was fully investigated.

Sometimes all the preliminary data required over a certain route may be obtained quickly by stadia methods, but some sort of a definite

line is required from which reliable conclusions can be drawn. Especially is this true in heavily timbered, hilly country, where the range of vision is very limited, and traveling on foot is excessively wearisome, and unless such a method is used some feasible route is very likely to be overlooked. Many times on these surveys the Assistant Engineer was sent back over parts of the line to hunt out something better, quite often with success.

In locating long grades, it was preferable to start at a summit and run down hill. With a little experience, the Assistant Engineer could make a sufficiently close estimate of the amount to allow for compensation for curvature, and could run his line accordingly. In the mountainous part of the country here described this compensation amounts to about 6 ft. per mile, equal to 0.12% grade and preliminaries, for a 1.0% compensated grade, run on an 0.88% straight grade, gave the desired information.

In case of the choice of two apparently equal routes, a location was made over each, and estimates were prepared for comparison before choosing a line. In following the larger watercourses, it was usual to locate a line on either side for purposes of comparison, and in order to determine the advisability of crossing from one side to the other either to get a better line or to block the country against rivals.

Some branch lines were located, and quite an extent of line was located and afterward abandoned by reason of change of plans, but at the conclusion the miles of line actually located footed up one-third more (this is practically the correct proportion, but it varied a little more or less on different divisions) than would be required on the entire layout. This extra location was partly for comparison, but largely for reasons mentioned later. In addition to this, many miles of paper location, not located on ground, were made for the same reasons.

It may appear to some that there was much unnecessary location and running of preliminary lines, but in rough country like this, and on work of this magnitude (in 220 miles of this line there were twenty-one tunnels, the longest being 4 000 ft., five viaducts from 400 to 1 000 ft. long, and more than 100 ft. in height, besides numerous other bridges), it is time and money well spent. In no other way can the exact data be gotten, and it leaves no question as to the available routes and the grades obtainable.

The locating engineer, or others on the ground, may feel certain that a line is not feasible, but it is hard to furnish proof, both to himself and to others who are interested, but who will never be over the ground, that such is the case, except by map and profile. Again, an apparently hopeless line may show up much better than expected, and *vice versa*. In this the writer must not be misunderstood to mean that anyone can get the best location if he only runs lines enough; such is far from the case; lines can be run indefinitely without securing the desired results, unless the proper judgment and knowledge are combined with them, the location being usually the most difficult problem of railroad work. What is meant is that it is necessary to run a sufficient number of lines, preliminary and location, to arrive at correct conclusions and to get the requisite exact data to prove the conclusions.

Topography, showing contours, houses, roads, watercourses, etc., etc., was taken on practically all lines. This was taken on 12 by 18-in. sheets. The transit line was plotted each day on the requisite number of sheets; a light pencil line, at right angles to the center line, was drawn through each station, for ease in plotting the topography; elevations were marked at each station; the stations where contours crossed the center line were determined from the profile and marked; connections to other sheets were shown, and then the sheets were ready for field use. The lines were plotted to a scale of 200 ft. to 1 in. The topography was plotted in the field. A hollow drawing-board, 18 by 24 in., was used. The sheet in use was tacked to the board, and the additional sheets were carried inside. A strap around the shoulders of the topographer served to carry the board, and formed a support while plotting (Wellington's method).

This method was preferred to any other; it is quicker; saves much copying and plotting; the work can be plotted better in the field, where everything can be seen at the time of plotting; and at night the Assistant Engineer has a finished map to look over and study. The topography was taken accurately by using a metallic cloth tape for distances and a hand-level for elevations. Only in this way can one get a projected location to correspond closely with the actual one. The topography was ordinarily taken for 300 ft. on each side of the center line; at particularly difficult summits or similar places a strip from 1 000 to 2 000 ft. wide was often shown, the necessary topography being obtained by auxiliary lines. The sheets were inked in each night.

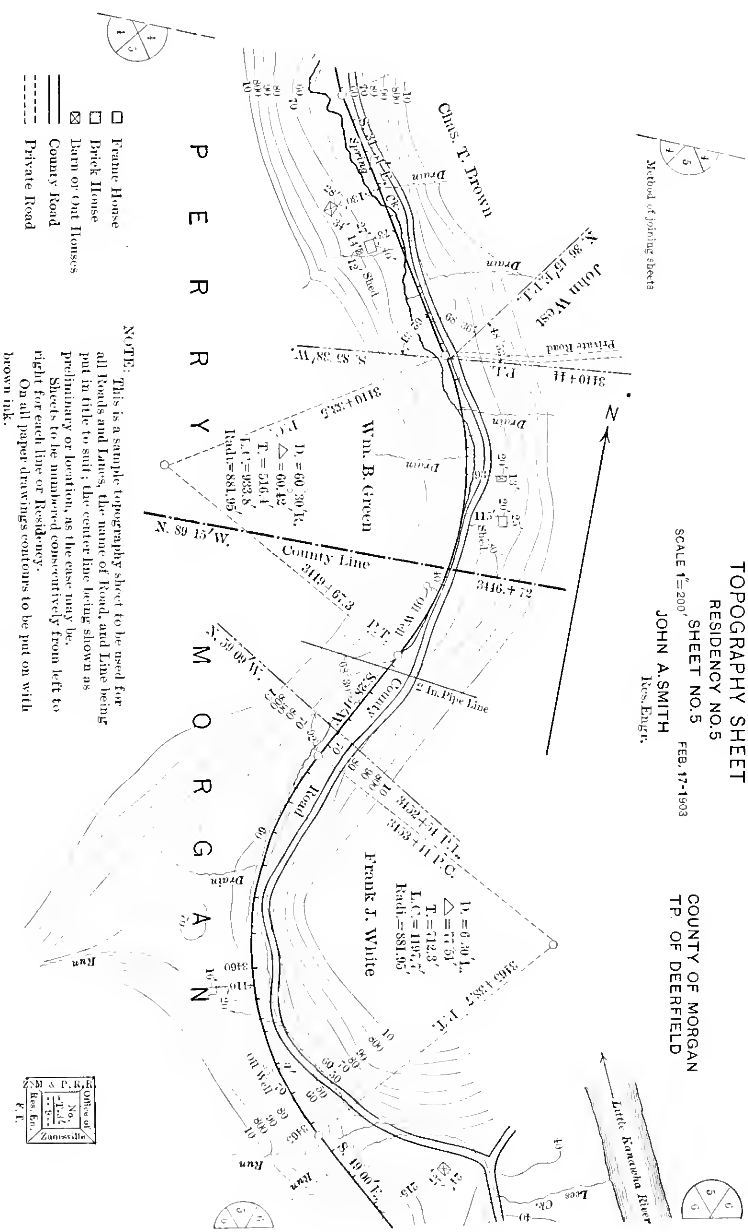

Z.M. & P.R.R.
TOPOGRAPHY SHEET
RESIDENCY NO.5
SHEET NO.5
SCALE 1"=200'
FEB. 17-1903
JOHN A. SMITH
Res. Engr.
COUNTY OF MORGAN
TP. OF DEERFIELD
N
Method of joining sheets
Chas. T. Brown
John West
Wm. B. Green
Frank J. White
PERRY
MORGAN
County Line
N. 89 15' W.
County Road
2 In. Pipe Line
Little Kanawha River
Lees Ck.
Private Road
Drain
Run
Shed
Oil Well
Spring
D. = 6° 30' R.
△ = 60.42
T. = 516.4'
L.C. = 933.8'
Radi. = 881.95'
D. = 6° 30' L.
T. = 712.3'
L.C. = 1197.7'
Radi. = 881.95'
3110+33.5 P.I.
3110+44 P.T.
3116.+72
3119+67.3
3152+51 P.I.
3153+41 P.C.
3163+38.7 P.T.
S. 83 38' W.
S. 28 15' W.
S. 19 00' E.
N. 59 00' W.
Frame House
Brick House
Barn or Out Houses
County Road
Private Road
NOTE: This is a sample topography sheet to be used for all Roads and Lines, the name of Road, and Line being put in title to suit; the center line being shown as preliminary or location, as the case may be.
Sheets to be numbered consecutively from left to right for each line or Residency.
On all paper drawings contours to be put on with brown ink.
Z.M. & P.R.R. Office of Res. En. Zanesville

FIG. 4.

To obtain a large general map showing all lines, the lines were carefully plotted on tracing cloth, the small sheets were fitted so as to make the center line on each sheet fit the center line on the tracing, and then the topography was traced. By this method any error in plotting or joining the small sheets was eliminated from the large map. This tracing, from which blue prints were made as required, was retained in the office, the sheets being used for rough work and field use.

The location was projected in pencil on sheets in the usual manner, care being taken to determine all angles between tangents by the calculated courses from each sheet, not by using a protractor, or by measuring intersection angles, although either might be used as a check. After the line was located on the ground, the location was accurately tied to the preliminary line and replotted on sheets; the pencil projection was then erased, if so desired.

In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections. The lines always varied more or less in their relative positions. Also, it was often found desirable to change the location at places, giving a corresponding change in the profile; for instance, it might be decided in the field that, owing to surface conditions, certain hills could be hit harder or perhaps avoided. The most frequent change was to put the line harder into steep side-hills. When projecting lines along such places, there is nearly always a tendency to fit the line too closely to the surface and not to allow enough cutting to put the roadbed on firm ground. Owing to these facts, it is usually better to have the first location projected and run in by the field engineer, who is most familiar with actual conditions; after that the revisions and necessary changes can be taken up by the higher officers. Projections made by anyone not thoroughly familiar with the ground should be used with caution. The best location cannot be obtained without the topography and a projected line; also, it cannot be obtained on the sheets alone, without a thorough and complete knowledge of the character of the country; the two must be studied together, the field engineer at all times corresponding freely and fully with the Chief Engineer and getting his ideas and instructions.

After the first location had been made, it was studied further in

the Chief Engineer's office; if any changes were desired they were taken up with the Assistant Engineer, usually by the Assistant Chief Engineer and the Assistant Engineer going over the ground together and there studying the question. After settlement, the location was taken as final for that route, and the maps and plans were brought up to date.

B. & N. R. R.

Profile of { "A" Preliminary / Projected Location. / "L" Location / Revised Location. }

from -------------- to --------------

Sta. ------ to Sta. ------

Date

Name of Engineer.

-------------- Chief Engr

B. & N. R. R.

Profile of Revised Location

on

Subdivision No. ------ Div. No. ------

Sta. ------ to Sta. ------

Date

------------ Chief Engr. -------------- Resident Engr.

B. & N. R. R.

Progress Profile.

Between sta. ------- & sta. ---------

Divn No. ------ Subdivn No. -------

Sect'n ------ to ------ inc.

Date

------------ Chief Engr. -------------- Resident Engr.

Note — Titles to be put in upper left hand corner of profiles.

FIG. 5.

Standards.—All curves of 3° or more had spiral approaches. These were allowed for in cross-sectioning by offsetting the slope stakes the required distance. The spiral was in standard form, so that, when the degree of curvature was known, the offset from the simple curve and the length of the spiral were obtained from tables. For simplicity and ease, all records, profiles, etc., were kept on simple curve data. When spiral curves came in tunnels, a special plan was made for each case,

showing the offsets from the tangent and the simple curve at every 10 ft. on the spiral, the alignment being kept on the tangent and the simple curve, and allowing the required offset in giving the widths for the tunnels. All grades were compensated 0.04′ per degree of curvature.

Vertical curves were inserted at all places when the change of grade was more than 0.1 ft. in 100 ft., a standard form being used in which the length of the vertical curve varied directly with the change of grade.

The usual weekly reports, maps, and profiles were sent in by the Assistant Engineers. Standard forms were used for all notes, maps, profiles, plans, and reports. The standard rules were first issued as typewritten copies, and standard forms were adopted from time to time as needed; later, they were compiled and issued in book form, being blue-printed from tracings, making 22 pages of general rules and information, together with 24 standard forms for notes and plans. It was found to save much work and time in the Chief Engineer's office to have all data in uniform shape; much correspondence was likewise avoided, as employees could readily find sizes, scales, etc., for plans, and also other information. Therefore they were not compelled to write to headquarters, and thus avoided the delay of two or three days required for letters to go and come.

A few of the standards used extensively on location are here shown.

Fig. 4 shows the standard topography sheet, the same form being used on preliminary and construction lines. After the line was finally determined, sheets were prepared showing that line only, and the title was made to suit the construction work shown.

The standard titles for profiles are shown by Fig. 5, and the standard titles for field books, by Fig. 6. The standard forms for transit and level notes are shown by Figs. 7 and 8, respectively. Fig. 9 shows the standard form for a location profile, and Fig. 10 shows the standard form for a situation survey for a bridge. When first prepared the structure was shown in pencil, and was not inked in until the detailed design had been made by the Consulting Bridge Engineers, A. P. Boller and H. W. Hodge, Members, Am. Soc. C. E. The standard form of report of openings required is shown by Fig. 11.

Cost.—The greatest number of miles of preliminary line run in one day by one party was 7, and of location, 4½. The location averaged slightly more than 1 mile per day per party, except on the Burnsville and Eastern and on the Buckhannon and Northern lines, where it

L. K. R. R.

Transit or Level } Notes on Prel. line "A" from Grafton to Moatsville, Sta. 0 to 750.

Date__________ Ass't Eng'r or Levelman.

Book No.________

L. K. R. R.

Transit or Level } notes on "L" Location via "A" Prel. from Grafton to Moatsville, Sta. 0 to Sta. 748

Date__________ Ass't Eng'r or Levelman.

Book No.________

L. K. R. R.

Transit or Level } notes on revised location on Subdivn No.____ Divn No.____ Sta._____ to Sta._______

Date__________

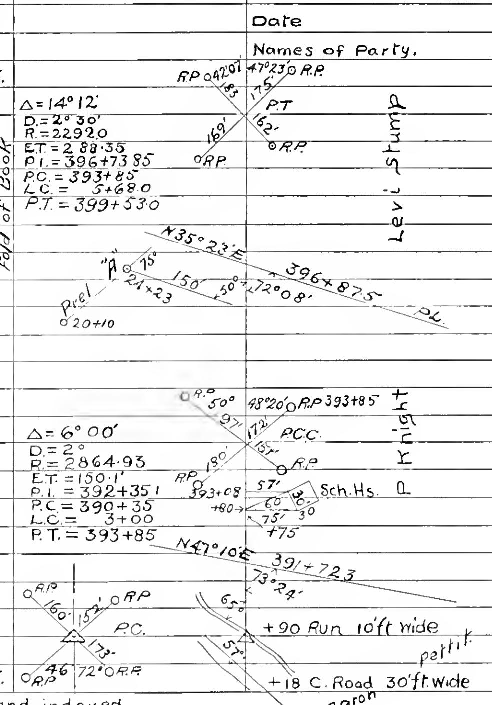
Book No.________ Resident Eng'r

Fig. 6.

TRANSIT

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	Deg Curve	Deflection.	Delta.	Calculated Bearing.	Magnetic Bearing.
401					
+50					
400 Tang 7465				S79°08'E	S78°45'E.
+53 P.T.		7° 06'			
399		6° 26'			
+50		5° 48½'			
398		5° 11'			
+50		4° 33½'			
397		3° 56'			
+50		3° 18½'			
396		2° 41'			
+50		2° 03½'			
395		1° 26			
+50		0° 48½'			
394		0° 11'			
+85 P.C.C.	2°30' L.	3° 00'	14° 12'		
+50		2° 39'			
393		2° 09'			
+50		1° 39'			
392		1° 09'			
+50		0° 39'			
391		0° 09'			
+85 P.C.	2° L.		6° 00'		
+50					
390				S.58°56'E	S.58°35'E.



Field books to be paged and indexed.

FIG. 7.

Level

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	Back Sight.	Height Inst.	Foresight.	Elevation.	Grade.	Cut or Fill.	
B.M.				100.00		B. M. № 10. on OAK. 20 ft. Left Sta. 910+50	Date.
Inst.	10.04	110.04					Names of Party.
910			5.2	104.8	97.00	C. 7.8 +35	HW= L.W.=
+40 Run			12.4	97.6	97.20	C. 0.4	+45 Run 10 ft. wide.
+90 Road			3.3	106.7	0/0 97.45	C. 9.2	+74 Road 30 ft.
911			2.5	107.5	+5/10 97.5	C. 10.0	11+05
T.P.			11.94	98.10			
Inst	0.72	98.82					
912			0.8	98.0	98.0	Grade.	
+50 P.C. 5 L.			1.2	97.6	98.15	F. 0.6	
913			5.3	93.5	98.30	F. 4.8	
+50			8.5	90.3	0/0 98.45	F. 8.2	
914			4.6	94.2	3/10 98.60	F. 4.4	
+50 P.T.			3.6	95.2	98.75	F. 3.6	
915							

FIG. 8.

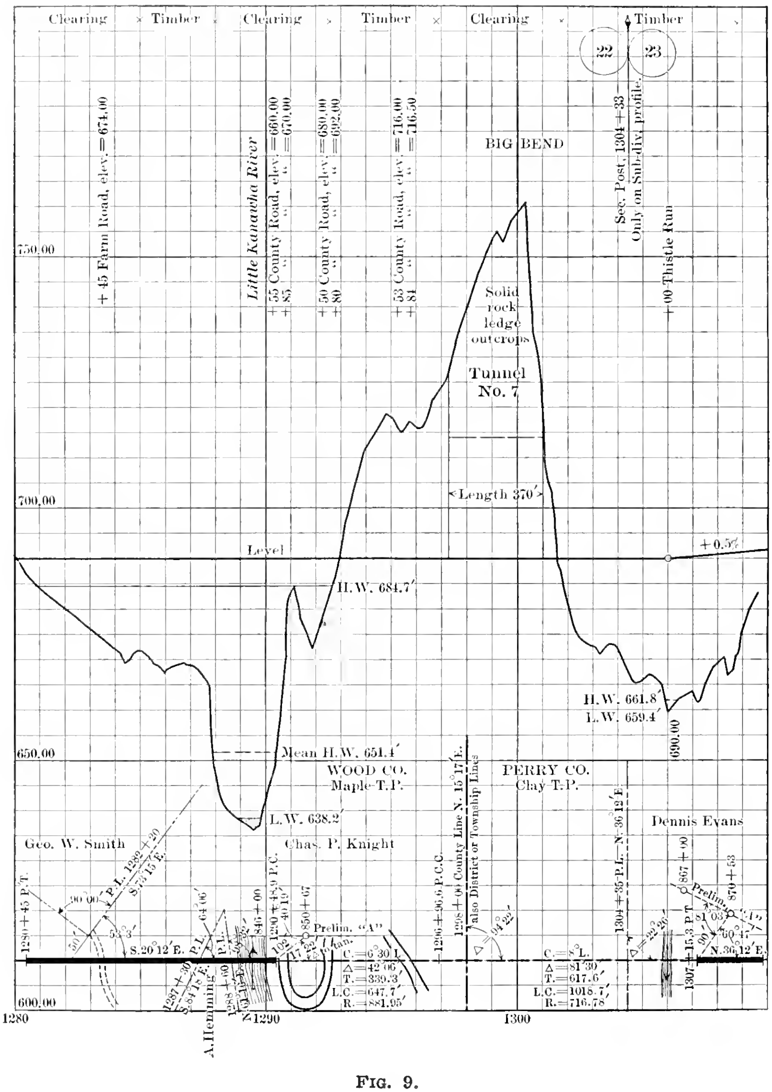

Clearing
Timber
Clearing
Timber
Clearing
Timber
22
23
+45 Farm Road, elev. = 674.00
Little Kanawha River
+55 County Road, elev. = 660.00
+85 " " = 670.00
+50 County Road, elev. = 680.00
+80 " " = 692.00
+53 County Road, elev. = 716.00
+81 " " = 716.50
BIG BEND
Sec. Post, 1304+33
Only on Sub-div. profile.
+00 Thistle Run
750.00
Solid rock ledge outcrops
Tunnel No. 7
Length 370'
700.00
Level
+0.5%
H.W. 684.7'
H.W. 661.8'
L.W. 659.4'
650.00
Mean H.W. 651.4'
WOOD CO.
Maple T.P.
PERRY CO.
Clay T.P.
L.W. 638.2'
Geo. W. Smith
Chas. P. Knight
Dennis Evans
1296+96.6 P.C.C.
1298+00 County Line N. 15° 17' E.
also District or Township Lines
1304+35 P.L. N. 36° 12' E.
S.20° 12' E.
Prelim. "A"
C.=6° 30' L.
Δ=42° 06'
T.=339.3'
L.C.=647.7'
R.=881.95'
C.=8° L.
Δ=81° 30'
T.=617.6'
L.C.=1018.7'
R.=716.78'
N.36° 12' E.
1307+15.3 P.T.
867+00
870+53
Prelim.
A. Hemming
600.00
1280
1290
1300

Fig. 9.

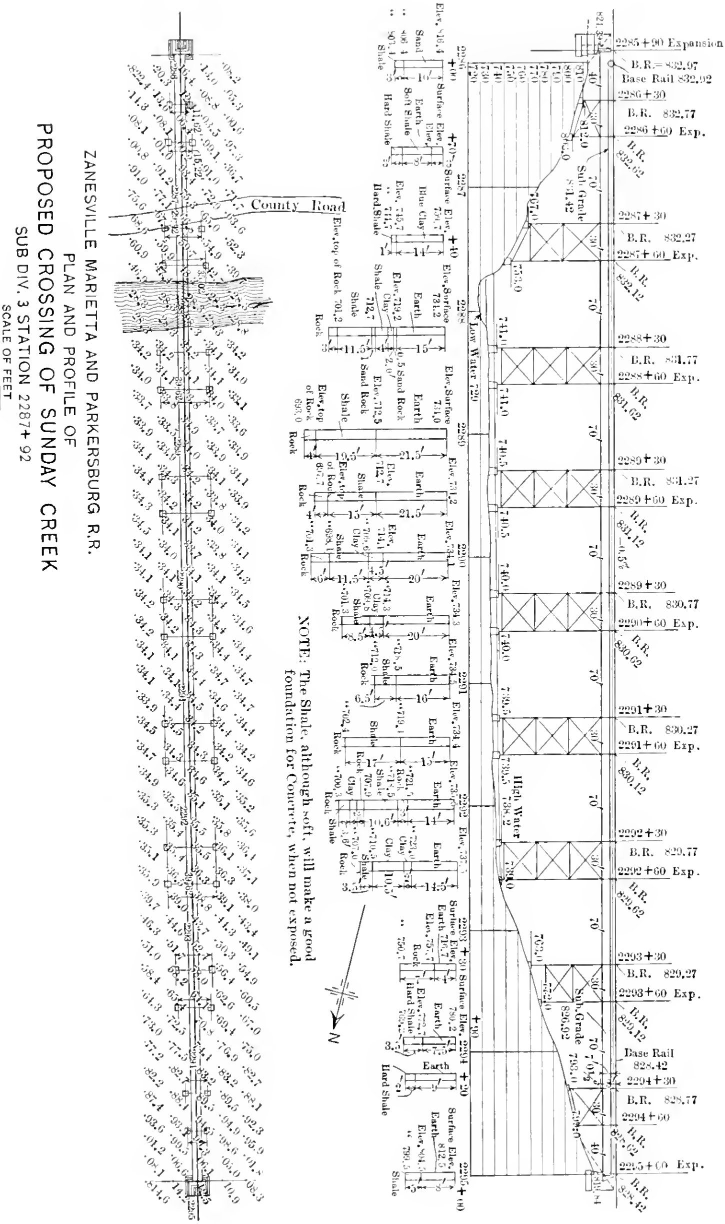

ZANESVILLE MARIETTA AND PARKERSBURG R.R.
PLAN AND PROFILE OF
PROPOSED CROSSING OF SUNDAY CREEK
SUB DIV. 3 STATION 2287+ 92
SCALE OF FEET
0 20 40 60 80 100
Chief Eng'r
County Road
NOTE: The Shale, although soft, will make a good foundation for Concrete, when not exposed.
Low Water 729
High Water 738.2
N

FIG. 10.

averaged ¾ mile. Stakes were set every 100 ft. on tangents, and every 50 ft. on curves. Special pains were taken with the instrument work and measurements, in order to avoid the chance of serious errors in the center line after construction commenced. The speed of location parties was usually limited by the amount of clearing that could be done, but the number of curves and the rough character of the ground were also large factors in limiting the speed.

Each party cost from $35 to $40 per day, being allowed all expenses in addition to salaries.

Table 1 gives the cost per mile of the completed surveys. It is to be noted that this is the total cost, and includes office rent, purchase of instruments and supplies, general expenses, all salaries, field expenses, and the preparation of final maps, plans, profiles, and estimates, with everything in readiness to make contracts for the line.

TABLE 1.

Company.	Amount spent.	Miles of Surveys:			Average cost per mile.	Average cost per mile of location.
		Preliminary.	Location.	Total.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
L. K. R. R.	$25 076.83	428.19	193.85	622.04	$40.31	$129.36
Z. M. & P.	19 812.77	509.03	105.23	614.26	32.25	188.28
B. & E. R. R.	20 466.68	241.75	113.70	355.45	57.58	180.00
P. B. & T. R. R.	6 651.98	84.56	38.17	122.73	54.20	174.28
B. & N. R. R.	19 249.94	162.51	151.29	313.80	61.34	127.23
Totals	$91 258.20	1 426.04	602.24	2 028.28	$45.00	$151.53

Column 7 gives the cost per mile of actual location, including preliminary lines. Columns 3 and 4 show that there were from 2 to 5 miles of preliminary lines run for each mile of location, except on the Buckhannon and Northern line. Table 1 also includes 302 miles of check levels, the cost being distributed among the various accounts. The data for the Parkersburg Bridge and Terminal line include surveys and soundings for the Ohio River Bridge. The cost per mile includes the topography on practically all lines, except on the Zanesville, Marietta and Parkersburg line, where it was taken only on the located lines.

The cost shown in Table 1, being the total charge against engineering from the inception of the project to the beginning of construction,

STANDARD FORM OF REPORT OF OPENINGS REQUIRED.

Z. M. and P. R.R.

List of Proposed Openings on Sub-Division No. 2. Sections 1 to 10 inclusive.

Sta. 1610 to 2138

Station.	Name of Opening.	Acres Drained.	Elev. of High Water	Elev. of Low Water.	Area Sq. Ft. High Water.	Proposed Opening.	Adopted Opening.	Remarks.
1611+75	Sunday Creek	Not Estimated; probably 60 Sq. mi.	738.2	729.0		Viaduct.	Definitely Determined by Higher Officer than Asst or Resident Engr.	See Situation Survey.
1614+15		5		730.0				Ditch to 1611+75
1815+00	Federal Creek	900	740.0	734.0	125	12' Arch		Co. Road has 14' Arch. Drainage Area Cleared
1992+60	Bill Run	300	742.0	739.0	45	D-4×6		Stream has rapid fall. Timbered
2001+15	Dry Run	120	743.0	742.0	20	6×8		Stream Discharges Much Driftwood.

Note.

No absolute formula used to determine size of opening; mostly determined by size of existing openings and Judgment of Engineer.

Following Formula used as basis (Myer's)

Area of Opening in Sq. Ft. $= C\sqrt{\text{Drainage Area, in Acres}}$.

Max. for $C = 4.0\pm$ Ordinary Min. $C = 1.0$

Min. so as not to give less than 1 sq. ft. per 12 Acres; for the small areas.

Where bridges are required, the opening being determined by economical design of structure.

John Jones
Resident Eng'r
Joy, Ohio.
May 21, 1903.

FIG. 11.

contains a few items which might well be charged to other accounts than location. Instruments purchased could be a credit; some elaborate property surveys and bridge surveys could be charged to construction, but they probably are not large enough to have much effect on the cost per mile. If taken into account, they would reduce the cost. The cost on the Little Kanawha and on the Burnsville and Eastern Division was increased considerably owing to much work being done during a bad winter, when the weather was very unfavorable. The cost on the Parkersburg Bridge and Terminal line was increased by a large amount of property surveying in the city, and by the surveys for the bridge.

The cost on all the West Virginia lines was increased by the immense amount of chopping and clearing necessary. When mountain laurel was encountered, all the axemen that could be worked could not keep a location party moving.

The influence of the weather is a large item in the cost of these surveys; a line run in the middle of winter may easily cost one-quarter more than if run during more favorable weather.

The average cost of one mile of preliminary or location survey, determined from a detailed study of the daily reports of field parties, of office work done, and similar data, is shown by Table 2 (to the nearest dollar), and is believed to be very close to the actual figures.

TABLE 2.

Company.	Average Cost of One Mile:		
	Of preliminary.	Of location.	Of location, including preliminary.
L. K. R. R.	$25	$74	$99
Z. M. & P. R. R.	23	79	102
B. & E. R. R.	35	105	140
B. & N. R. R.	31	94	125

The figures in Table 2 include all expenses, as in Table 1.

Table 1 shows a large variation in the cost of surveys on different divisions, the cost varying from $128 to $188 per mile, with an average of $151. On the assumption that lines located for comparison or similar purposes should be included in the average, one-third should be added to these amounts, as previously noted; the cost would then be as follows:

Low	\$171 per mile.
High	251 " "
Average	202 " "

Throwing out of the account the mileage of abandoned lines, branch lines, etc., and charging the entire cost to the main line, terminus to terminus, would give $\frac{\$91\,258.20}{328} = \$278.23$ per mile, which would be rather expensive. This, however, is not a fair assumption, and should not be considered, because many miles of lines not needed to determine the main line were located for other reasons and purposes. Therefore, the plan of throwing out only duplications, for comparisons, as shown in the preceding paragraph, gives the correct average cost per mile for the development of the country, including actual comparative locations where needed. It should also be borne in mind that a large proportion of this duplication was necessary, owing to the laws of West Virginia, which require an actual line, located on the ground, and a complete map and profile of that line to be filed with the Secretary of State, and at the county seat, before a railroad company has any rights, of priority or otherwise, to that route or line. This required complete locations for all proposed branch lines, a large number of which were located, and also a complete location over any route for which it was desired to obtain rights. For these reasons, the lines located account for the excess of the mileage over the actual length of the main line.

On the basis of Table 2, it may be assumed that, where the route has been previously determined within such narrow limits that the preliminary and location lines are of equal length, the surveys will cost from \$100 to \$140 per mile. This is borne out by the results on the Buckhannon and Northern line where the location and preliminary lines were practically equal and the cost was \$127 per mile.

These two statements may be combined and put in the following form:

To locate one mile, including an equal length of preliminary lines, cost from \$100 to \$140; average........	\$115
To locate one mile, final location, including from two to five times as great a length of preliminary lines, cost from \$128 to \$188; average........................	151

To locate one mile, final location, including from two to five times as great a length of preliminary lines, and one-third of a mile of location for comparison, cost from \$171 to \$251; average........................ \$202

A tabulation of the mileage of the Buckhannon and Northern line, with reference to the actual length of line to be built, and showing how the results agree with the averages deduced from Table 1, is as follows, the Buckhannon and Northern line being used because the conditions there make it the best average of "all conditions" encountered on the various lines.

Total miles located..........................		151.29
Miles of main line contracted for..........	80	
Miles of main line not contracted for......	4	
Miles of connecting line located, but which may or may not be built, about........	26	110.00
Making actual miles......................	110	
Leaving duplications, comparisons, etc.............		41.29 miles.

110 miles cost \$19 249.94 = \$175 per mile.

Results.—The results obtained by studying and surveying the neighboring country, will amply justify the initial cost, and, without doubt, the final results will prove that it is cheaper in the end. A few thousand dollars judiciously spent on surveys will be more than made up for in construction, not only in the actual cost, but in a better line.

That the desired results seem to have been obtained, may be shown briefly as follows:

1.—Easiest grades: 0.5% west, 1.0% east (all the latter in the 25 miles near the eastern end), other roads using 1.5 per cent.

2.—Curvature: 8° maximum, as against 12° on other roads.

3.—Short line: This is seen best on the map. The Little Kanawha line is 31 miles shorter than the river, the only water route. The Ohio line is 16 miles shorter than the existing lines between termini.

4.—Occupation of critical points: Another line, since built, through the Burnsville and Eastern Division (the Burnsville and

Eastern line never having been constructed) and a few miles on Little Kanawha Division, had to locate on and occupy said lines for about 15 miles, although using 1.5% grades and 12° curves. It could not have been built for its present cost without making arrangements with the Burnsville and Eastern and Little Kanawha Railroads.

5.—The definite places previously determined on were, of course, reached.

6.—It being decided, after many surveys—by which the advantage of this line proved so apparent—to build the line as described, no complete estimate of the cheapest possible line in first cost was prepared, but a sufficient number of estimates were made to show that the line as located would certainly fulfill this requirement.

All the work was carried out under S. D. Brady, M. Am. Soc. C. E., then, as now, Chief Engineer of all these companies, who gave much personal attention to the work, and to whom the writer is indebted for assistance and information in preparing this paper, and for permission to publish the maps and costs.

The writer was Assistant Engineer on the Little Kanawha Division, in charge of location; on its completion he became Assistant Chief Engineer of all the companies, both on location and construction.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.

By Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.

To be Presented June 2d, 1909.

The principal tests described in this paper were made on the following compression pieces: (*a*) a steel column (called Column No. 1) built up of angles, plates, and lattice bars, all the parts being light; (*b*) four wrought-iron bridge posts which had seen long service in a bridge truss; and (*c*) three posts and a top chord in a railroad bridge under service. The tests of (*a*) and (*b*) were made in a testing machine; for (*c*) a locomotive and cars formed the load. Auxiliary tests were made on lattice bars and other parts.

These tests were taken up with a view of determining experimentally: (1) something of the variation of stress, both longitudinal and lateral, throughout the channel members of the compression piece; (2) something of the amount and distribution of stress in the lattice bars of columns, and the action of similar bars under separate test; and (3) the general relation between the component parts and the column as a whole. The slenderness ratio of the columns, $\frac{l}{r}$, ranged from 38 to 66. It will be seen that instead of following the more

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

common effort to determine the effect of length of column, emphasis was placed on measuring the distribution and range of stress over the various parts of the column.

It is unnecessary to state that built-up compression pieces (whether long or short) are not perfect, the natural imperfections of the component parts being increased in the process of fabrication. To non-homogeneity of structure and lack of straightness in the component angle or channel are added such further imperfections as kinks and eccentric connection of parts, which go to increase the opportunities for local flexure in the component parts and for flexural stresses in the column as a whole. An attempt has been made in these tests to measure the deformations in the presence of such conditions, and to find the general distribution of stress. In view of the many limitations surrounding such tests, the results are to be taken as suggestive and qualitative, and not as exact determinations.

Results of Tests.

The following observations on the results are given here with a view of showing the trend of the tests:

I. Variation of Stress Throughout the Length and the Cross-Section of the Channels and of the Column as a Whole.—Channel members show evidence of considerable local flexural action, such as may be produced by lack of straightness or by any method of applying the load eccentrically. This is especially true in the flimsier column.

The condition of flexure varies markedly throughout the length of the member, in some cases the maximum compression in one cross-section being at the extreme fiber on one side of the channel, and in a near-by section the other side of the channel showing the excess of stress.

There were indications of sudden changes in the relative amount of stress carried by the two channels at near-by sections, indicating general flexure of the column.

The measurements made indicate in a number of cases stresses in the extreme fiber from 40 to 50% in excess of the average stress, and in some cases even higher.

The amount of eccentricity necessary to account for the increase of stress found in individual channels, based on lack of straightness and the ordinary theory of flexure, is relatively small.

II. Stress in Lattice Bars.—The amount of deformation observed in lattice bars is relatively small.

The measurements indicate a stress in the lattice bars which would be produced by a transverse shear equal in amount to 1 to 3% of the applied compression load, or to that produced by a concentrated transverse load at the middle of the column length equal to 2 to 6% of the compression load.

III. Tests of Lattice Bars.—Tests of lattice bars for load-carrying capacity show that the usual form of bar is a very inefficient compression member when loaded eccentrically through a riveted connection.

Under common conditions of loading, the maximum fiber stress may be as much as three times the average stress.

In the compression tests of single lattice bars, the ultimate strength was in no case as much as one-half of the elastic limit of the material.

IV. Relation between Component Parts and the Column as a Whole.—It is a question whether the component members of a built-up column act together to form an integral compression piece, especially in resisting oblique distortion.

The distribution of stress under working loads, and even up to incipient failure, may be quite different from that which exists when the column becomes crippled. This is due to the yielding of the more strained parts after the yield point is reached at any fiber, and the consequent redistribution of stress.

No relation has been found between the stresses actually observed and the stresses computed by column formulas.

The sudden failure of a test column at a relatively low load by buckling of lattice bars is accounted for when the amount of transverse shear developed in other test columns and the strength of lacing found in lattice-bar tests are taken into consideration.

Laboratory Tests of Columns.

Description of Columns.—One steel column and four wrought-iron columns were tested. The steel column was built especially for the purpose of this test by the American Bridge Company at the Lassig plant. The wrought-iron columns were halves of bridge posts taken from an old bridge of the Chicago, Burlington, and Quincy Railroad.

COLUMN NO.1

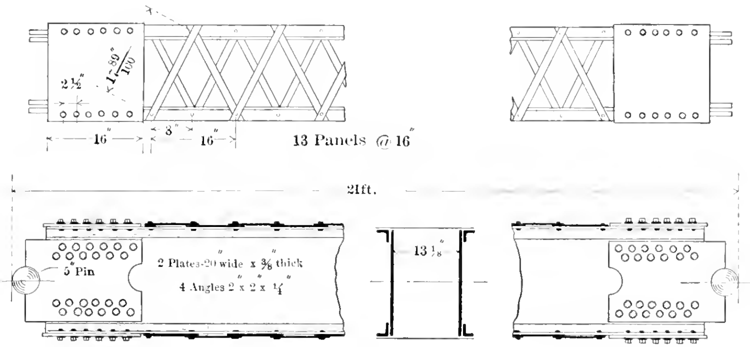


FIG. 1.

WROUGHT-IRON COLUMNS

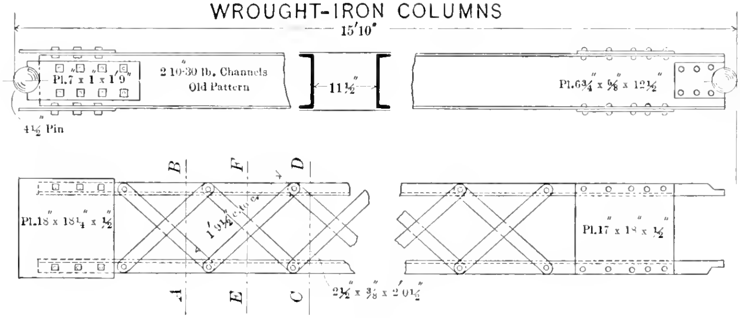
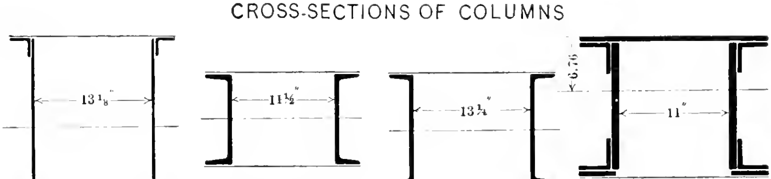


FIG. 2.

CROSS-SECTIONS OF COLUMNS



Column No.1
2 Plates 20" x ⅜"
4 Angles 2" x 2" x ¼"
Area = 18.76 sq.in.

Wrought Iron Posts
Nos. 2,3,4, and 5
2-10" Channels
Old Model
Area = 17.61 sq.in.

Posts
White Heath Bridge
2-12" Channels
Old Model
Area = 12.02 sq.in.

Upper Chord
White Heath Bridge
2 Plates 14" x 11/16"
4 Angles 3½ x 3½ x 7/16"
1 Cover-Plate 21" x 9/16"
2 Flats 4" x ¾"
Area = 48.67 sq.in.

FIG. 3.

TABLE 1.—DATA OF COLUMNS.

Column designation.	Column No. 1.	Wrought-Iron Posts. Nos. 2, 3, 4, and 5.	Wrought-Iron Posts. Retests 2*a* and 4*a*.	White Heath Bridge. Posts $U_3 L_3$ N., $U_3 L_3$ S.	White Heath Bridge. Upper chord $U_3 U_4$ south.
Area of column section, in inches	18.76	17.64	17.64	12.02	48.67
Length, center to center........	21 ft.	15 ft. 10 in.	14 ft. 7 in.	25 ft.	19 ft. 10 in.
End conditions..................	Pin parallel to lacing.	Pin parallel to lacing.	Pin parallel to lacing.	Lower end pin, upper end riveted	Riveted.
$\frac{l}{r}$, axis parallel to lacing	37.8	43.5	40.1	66.1	40.7
$\frac{l}{r}$, axis perpendicular to lacing.	37.2	41.2	38.0	41.0	29.6
$\frac{l}{r}$ of each flange member, axis perpendicular to lacing for full length of column...	593	400	367	416	....
$\frac{l}{r}$ of each flange member, axis as before, for distance between adjacent lacing rivets.	37.7	33.7	33.7	22.2	....
Lattice bars, dimensions of section, in inches................	1 by $\frac{1}{4}$ in. and $1\frac{1}{4}$ by $\frac{7}{16}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{4}$ by $\frac{3}{8}$ in.	$2\frac{3}{4}$ by $\frac{3}{8}$ in., on bottom.
Kind of lacing..................	Single.	Double.	Double.	Double, riveted at crossing.	One cover-plate.
Angle of lattice bar with axis of column..........	63° 30′	45°	45°	45°	45° on bottom.

Table 1 gives the dimensions of the columns. The details are shown in Figs. 1 and 2; and Fig. 3 shows the cross-sections. The steel column (Column No. 1) was designed with relatively thin component parts and relatively large outside cross-section dimensions. It will be seen that this column is not so stocky as the usual bridge column. This section was chosen because it seemed to offer better opportunities under the conditions of the test for the study of the distribution of stress, longitudinal and lateral, and especially of the stresses developed in the latticing, than a less flimsy column. In the earlier tests, the lattice bars of Column No. 1 were fastened in place with turned bolts in reamed holes so that one set of bars could easily be replaced by another. After several tests were made with the lacing thus fastened, the bars were riveted in place for the later tests. The turned bolts in reamed holes apparently held the column together in the column tests as well as did the rivets. The proportions of the wrought-iron bridge columns were in no way unusual, and the posts represented good practice at the date of construction of the bridge. The columns

became available through the replacing of the bridge with a heavier structure; they were apparently in good condition.

All the columns were pin-ended, the pin being parallel to the plane of the lacing. The details of the ends are shown in Figs. 1 and 2. The wrought-iron posts were cut in two. One end was left as used in the bridge, and bearing plates and batten plates were bolted to the other end.

Procedure of Tests.—The method used in studying the distribution of stress was to measure at various parts of the column, by means of extensometers, the deformation over short distances, and its variation over a section perpendicular to the axis of the column. The extensometers were placed outside of the flanges of one of the channels in such a way that the shortening at any point of the cross-section of the member could be determined on the hypothesis that in this part of the channel a plane section before loading remained a plane section after the load was applied. It is seen that this hypothesis is not dependent upon the integrity of the column as a whole. A similar arrangement was made in observing deformations in lattice bars. The position of the instruments is shown by Fig. 1, Plate XI. Two sets of instruments were used, and sometimes more. Generally, they were placed on the front and back flanges of one channel and then on the other, but sometimes on the front or back flanges of two channels at the same time. This procedure necessitated the removal of the instruments to new positions after several applications of the load, and, in the complete test of any column, involved several hundred applications of the load.

In all the tests to determine the distribution of deformations, care was taken not to exceed the elastic limit at any point. Under these conditions, it is considered, for the purposes of the discussion, that the stress developed at any point is proportional to the deformation. The ratio of the intensity of the stress to the deformation per unit of length, therefore, will be taken to be equal to the modulus of elasticity of the material.

Extensometers.—In the observations of deformation in the channel members of the columns, the extensometers used in most of the tests were Ames test gauges mounted on suitable frames. These gauges were graduated to $\frac{1}{1\,000}$ in., and were read by estimation to $\frac{1}{10\,000}$ in. These instruments magnify the change of length by means of a clockwork device rotating a hand on a dial.

Fig. 4 shows the method of attachment of these extensometers to a column. The gauged length was usually 8 or 9 in. As will be seen, the extensometers measured the change of length between points slightly outside of the members tested. From the actual instrument readings were computed the deformations and stresses at the extreme fibers of the flanges of the channels by the ordinary method, which assumed a rectilinear distribution of deformation and stress.

The type of instrument used is light, simple, and convenient, and, under very severe usage in other tests, it had shown itself to be durable. The limits of accuracy of the instrument were fairly well determined by careful calibration. While the maximum errors possible are large, it gives fairly trustworthy results under careful manipulation, and any available instrument of greater precision would be too bulky or too likely to have its parts deranged under the conditions of column tests, especially in field tests, to be entirely satisfactory.

ATTACHMENT OF EXTENSOMETERS TO CHANNELS

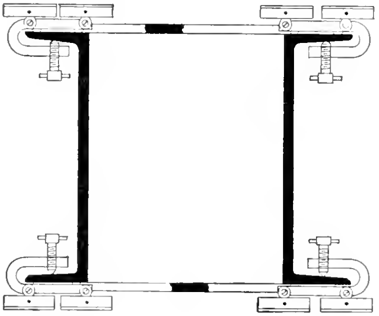


Fig. 4.

The accuracy of all these instruments was tested by comparison with a Brown and Sharpe micrometer acting through a 10 to 1 lever. Basing judgment on the maximum deviation observed in the calibration, and on the smallest deformations measured in any test of columns, it is felt that the error in stress determination for the channel members is generally less than ± 10%, and that it is well within this limit for most of the determinations in the laboratory tests. This general limit of accuracy is corroborated by a comparison of the average stresses on the center of gravity of the flange members with each other and with the known average stress. To those accustomed to greater precision in calculations and to the greater refinement of most laboratory tests, this limit of accuracy may seem crude. Having in view that the purpose of the tests was qualitative rather than quantitative, and considering the great variation in the distribution of stress found in the columns, and the general consistency of the results, it may be considered that the instruments were satisfactory.

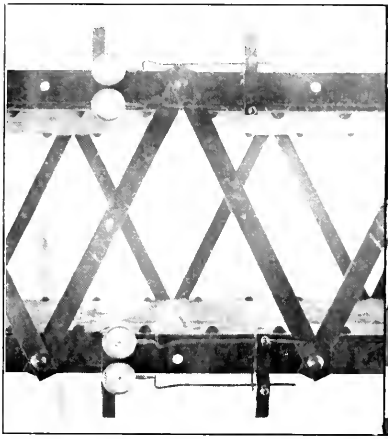


Fig. 1.—Extensometers in Place.

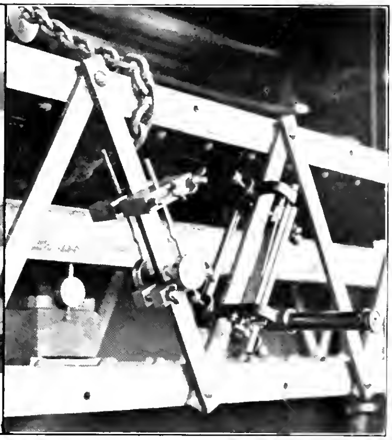


Fig. 2.—Attachment of Instruments in Cross-Bending Test of Column No. 1.

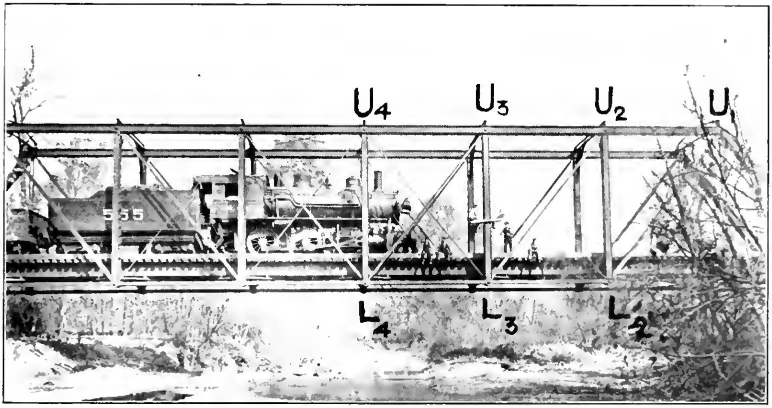


Fig. 3.—White Heath Bridge and Test Train.

In several of the tests, both in the laboratory and in the field, extensometers of the Johnson type were also used, but it was found that the results were not trustworthy, and no account of these measurements is given here.

The Ames test gauges were also used to measure deformations in the lattice bars of all columns but No. 1. On account of the very low stresses in the lattice bars, it is felt that the stresses determined in them in this way may be in error by ± 20%, or even more. In Column No. 1, a Ewing extensometer, reading to $\frac{1}{50\,000}$ in. was used, and the stresses in the lattice bars of this column are judged to have been determined with an error in any case of not more than ± 10 per cent.

Testing Machine.—The testing machine used was the Riehle, 600 000-lb., vertical, screw-power testing machine in the Laboratory of Applied Mechanics of the University of Illinois. This machine has a clear space of 36 in. between screws, and will take in compression specimens 25 ft. long. The speed of head used in these tests was in nearly all cases 0.4 in. per minute. This machine is equipped with massive guide frames which take the very heavy side thrust which occurs on the cross-head when a specimen is tested under oblique load. These guide frames are entirely independent of the weighing mechanism.

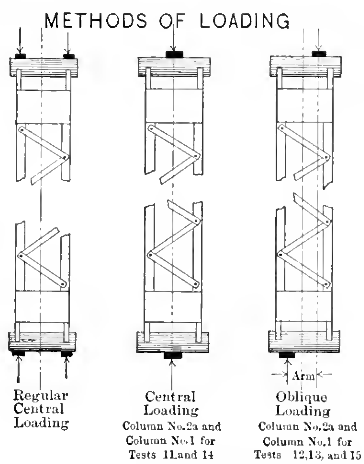


FIG. 5.

Method of Loading.—Two methods of loading were used, central and oblique. In all cases the load was applied at the ends of the column through the pins, and in a plane at right angles to the web and passing through the nominal axis of the column. In central loading, the pin was adjusted to an even bearing on the machine, the object being to apply the load equally to the two webs of the column. In oblique loading, the pin was supported on a narrow block, as shown in Fig. 5, in such a way as to secure a given eccentricity. The center of the block was taken as the point of application of the load. This is approximately true, the error probably not being greater than

¼ in. In two tests with Column No. 1, the point of application of the load is uncertain, as by an oversight the two bearing pieces used on each pin were placed unsymmetrically with respect to the axis of the column. It seems probable that the loading was nearly central.

Routine of Tests for Stress Distribution.—In a test for stress distribution, the column was placed in the machine and a light initial load was applied. The extensometers were then attached in position to measure the deformation at some point of the column, and an initial reading was taken. A known load was applied by the testing machine and the instruments were read again. The load was then released to its initial value and another application of the load was made. If the second readings did not exactly check the first, further applications of the load were made. In cases where the observed deformations were large or seemingly abnormal, the test was repeated at another time, and in some cases as many as ten observations were made on the same gauged length. In some of these cases the instruments were reset, their places being exchanged. The instruments were next attached in a new location, and the process was repeated. Thus the stress distribution in various parts of the column was finally determined. The load generally used in the laboratory tests was 10 000 lb. per sq. in. of section of the column in excess of the initial load.

Results of Tests for Stress Distribution in Channels.—Tables 2 and 3 give results of the tests to determine stress distribution and variation in the flange members found in twelve of the column tests. The stresses given are calculated from the observed deformation, using for the modulus of elasticity 28 000 000 lb. per sq. in. for steel and 26 000 000 lb. per sq. in. for wrought iron, these values checking closely with the total shortening of the columns and with the average deformations observed throughout their length. As heretofore described, the stress noted is the average over a space of 4 or 4½ in. on either side of the point indicated. Any lack of agreement between the average stress on the center of gravity of the flange members and the average stress for the load applied is probably due principally to instrumental errors.

Figs. 6, 7, and 8 show graphically the stress distribution and variation. The full line gives the stress at the west side (front) and the dotted line at the east side (back).

TABLE 2.—Stresses in Column No. 1.

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
						Test No. 1.						
1	8 100	10 200	10 600	12 200	10 000	9 700	6 500	8 400	8 600	9 600	10 900	11 300
2	9 700	10 100	10 100	8 100	9 600	10 000	11 900	11 500	11 600	8 900	10 500	11 100
3	7 500	10 200	11 000	11 800	10 000	9 700	11 500	9 800	9 800	9 000	10 300	11 000
4	10 500	10 000	10 000	10 500	11 000	11 000	9 000	8 600	8 600	10 800	10 100	10 500
5	9 600	9 100	9 100	10 900	9 700	9 500	10 300	8 500	9 700	8 800	10 100	10 600
6	9 900	11 000	11 100	10 000	9 600	9 700	9 400	8 400	8 100	9 000	9 200	9 100
7	10 000	7 600	6 900	11 000	10 000	9 900	9 000	9 400	9 900	9 200	10 000	10 400
8	8 100	8 900	9 000	10 800	11 300	11 500	12 700	10 500	10 100	12 200	10 100	10 300
9	10 000	11 000	11 100	9 400	11 000	11 000	14 500	10 000	9 000	9 000	9 200	9 200
10	14 000	10 000	9 300	14 300	11 800	11 000	5 400	8 900	9 300	7 200	9 600	10 000
11	8 600	9 500	9 600	10 000	9 800	9 600	9 000	8 800	8 400	9 800	11 000	11 400
12	12 300	9 200	8 700	9 100	9 400	9 800	10 800	9 300	9 000	10 700	9 600	9 500
13	.....	.....	.....	8 700	8 800	9 000	7 800	9 000	9 200	.....	.....	.....
						Test No. 2.						
6	11 500	12 100	12 100	11 500	10 000	9 500	9 100	9 000	9 300	5 800	7 500	7 700
7	12 200	12 600	12 700	14 600	13 500	13 300	5 900	7 500	8 000	8 300	9 500	9 500
8	11 300	12 500	12 800	12 400	12 500	10 600	7 800	8 200	8 300	7 500	10 300	10 700
9	14 800	13 300	12 700	9 500	10 500	10 600	11 800	7 300	7 000	10 300	8 400	8 300
10	9 000	10 200	10 400	16 200	10 500	9 600	7 800	10 500	11 400	9 000	8 400	8 500
11	15 200	10 200	9 700	13 700	11 600	11 500	7 600	8 100	8 300	5 000	6 300	8 600
12	10 300	11 900	12 300	12 700	11 600	11 500	9 400	8 200	7 700	7 000	7 500	9 700
13	13 200	11 800	11 700	9 900	10 500	10 800	7 500	8 600	9 100	6 300	6 300	8 400
						Test No. 3.						
1	.....	.....	.....	13 800	10 400	9 500	7 500	8 800	8 900	.....	.....	.....
2	8 900	10 600	10 900	8 800	10 400	10 600	8 300	9 300	9 400	10 000	9 700	9 700
3	10 500	9 800	9 800	8 700	9 000	9 200	10 400	8 700	8 700	11 900	10 500	10 300
4	9 800	10 800	11 000	11 100	10 700	10 700	8 300	8 300	8 500	13 300	12 600	12 200
5	10 500	10 000	9 800	11 200	10 700	10 700	9 000	10 000	10 100	9 300	9 500	9 400
6	11 300	12 300	12 500	10 900	10 500	10 700	10 400	9 700	9 500	7 500	9 400	9 600
7	10 400	10 700	10 700	11 900	11 000	10 800	9 200	10 000	10 000	8 400	8 800	8 800
8	12 100	10 200	10 200	12 100	13 000	13 100	8 500	9 300	9 400	7 300	9 000	9 400
9	7 600	9 900	10 200	10 500	10 900	10 800	14 400	9 300	8 400	10 900	10 600	10 700
10	10 500	12 000	12 100	14 200	11 000	10 500	7 200	10 400	10 900	7 800	8 700	8 700
11	14 200	10 500	9 700	7 500	8 500	8 600	8 300	9 400	9 300	7 000	9 400	9 800
12	10 400	12 000	12 200	6 500	7 500	7 800	11 000	9 300	9 000	7 900	9 100	9 400
13	12 600	9 800	9 500	8 300	8 400	8 200	7 200	9 800	10 200	11 000	10 800	10 900

TABLE 2.—STRESSES IN COLUMN NO. 1—(*Continued*).

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.

TEST NO. 4.

1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2	11 300	10 800	10 700	10 500	11 100	11 200	6 100	3 000	2 500	8 900	10 200	10 600
3	10 000	11 600	11 600	10 700	11 100	11 100	8 400	8 500	8 500	12 000	11 300	11 100
4	11 200	12 300	12 500	9 900	10 900	11 000	7 600	9 400	9 700	7 900	9 300	9 500
5	.....	.....	.....	12 800	11 100	10 700	.....	.....	.....	10 700	10 200	10 100
6	10 100	5 400	4 500	12 800	10 700	10 300	6 500	8 900	9 500	.....	.....	.....
7	8 000	10 600	11 000	10 200	9 700	9 700	.....	.....	.....	8 700	8 590	8 600
8	9 800	12 000	12 400	9 700	10 000	10 100	9 000	9 400	9 400	10 300	11 300	11 400
9	11 500	10 600	10 400	.....	.....	.....	8 500	10 700	10 900	7 900	11 100	11 500
10	8 600	8 300	8 100	6 800	8 700	9 100	10 800	7 500	7 000	9 300	10 700	11 000
11	.....	.....	.....	.....	.....	.....	9 500	9 000	8 900	13 000	12 700	12 600
12	10 900	10 400	10 300	10 700	9 900	9 600	.....	.....	.....	.....	.....	.....
13	11 000	10 700	10 700	11 100	10 600	10 500	7 300	9 800	9 600	12 700	11 900	11 800

TEST NO. 5.

½	12 100	10 400	9 300	11 000	9 500	8 700	9 700	10 500	10 900	10 400	10 800	10 900
1	9 800	11 100	11 800	10 000	10 200	10 500	12 200	10 400	9 300	13 500	11 300	10 400
1½	7 900	8 500	8 600	5 800	8 200	8 900	7 300	8 400	8 900	7 400	7 900	8 300
2	11 800	11 400	11 100	10 900	10 600	10 600	8 500	8 600	8 700	9 800	10 200	10 400
2½	10 700	11 000	11 100	10 800	10 200	10 200	11 800	11 900	11 800	9 700	9 700	9 600
3	5 800	7 500	8 500	7 800	8 000	8 500	8 300	8 500	8 600	7 700	9 100	9 700
3½	8 000	8 100	8 500	7 400	8 100	8 500	7 800	8 600	9 000	7 500	7 600	7 700
4	15 400	13 000	12 400	13 500	13 000	12 800	11 600	11 800	11 800	9 900	9 900	9 900
4½	12 200	10 700	10 300	12 200	10 200	9 200	9 900	9 900	9 800	9 800	9 800	9 700
5	9 400	9 500	9 600	11 000	10 100	9 600	9 700	9 500	9 500	8 900	9 800	9 900
5½	9 900	10 700	11 100	10 500	10 900	11 000	10 900	9 100	8 300	8 700	8 800	8 900
6	7 400	7 300	7 200	8 100	9 200	9 700	13 800	11 900	11 100	11 100	10 600	10 200
6½	10 900	10 200	9 700	10 200	10 300	10 500	8 600	8 900	8 900	8 500	8 600	8 600
7	12 100	11 700	11 400	11 400	10 900	10 400	8 300	8 800	8 900	5 900	7 900	8 700
7½	9 900	9 200	8 900	9 600	9 400	9 500	6 500	7 600	8 300	4 600	6 400	6 900
8	9 800	8 600	8 300	8 100	9 700	10 400	11 400	9 500	8 500	12 700	10 800	10 100
8½	9 200	9 700	10 000	9 700	9 500	9 300	20 200	15 400	13 100	15 400	14 000	13 200
9	6 500	7 600	8 300	9 700	9 700	9 900	12 900	13 200	13 100	9 400	9 900	10 200
9½	10 000	10 400	10 600	12 900	9 500	8 100	4 600	7 800	8 900	4 600	7 300	8 500
10	16 400	12 900	11 200	17 000	14 400	12 900	8 000	8 600	8 700	6 800	8 000	8 600
10½	11 300	10 700	10 400	9 900	10 000	9 700	10 500	11 100	11 300	7 400	8 900	9 600
11	7 000	8 400	8 900	8 100	9 100	9 600	12 600	11 100	10 800	9 800	10 800	11 000
11½	9 700	8 900	8 600	11 900	10 500	9 700	8 200	11 100	11 700	8 900	9 700	9 800
12	14 000	12 400	11 600	11 900	11 000	10 400	6 900	7 500	7 800	8 100	8 800	9 400
12½	9 700	8 900	8 600	10 500	12 100	12 700	10 800	11 800	12 400	11 900	11 000	10 800

TABLE 3.—Stress in Wrought-Iron Bridge Posts.

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Column No. 2, Test No. 6.												
1	4 600	6 500	7 300	6 400	7 900	8 500	4 300	8 300	9 700	5 000	6 800	7 400
2	7 300	7 300	7 300	8 300	8 400	8 400	9 800	9 000	8 800	9 000	8 600	7 600
3	5 500	5 900	6 000	7 100	7 800	8 000	7 700	8 800	9 200	6 700	7 600	7 900
4	6 200	6 100	6 200	6 100	6 900	7 100	8 400	8 800	9 000	8 200	8 000	8 000
5	7 200	6 500	6 100	8 800	8 600	8 500	7 500	8 800	9 300	8 700	7 800	7 500
6	6 800	6 900	6 900	8 400	8 400	8 400	7 100	7 600	7 800	6 900	7 200	7 400
7	7 500	7 900	8 000	7 100	7 400	7 500	7 500	8 200	8 500	7 500	7 400	7 300
8	6 400	7 400	7 800	6 200	7 200	7 600	7 700	7 900	8 000	6 600	6 400	6 100
Column No. 3, Test No. 7.												
1	9 300	9 000	9 000	8 400	9 500	10 000	10 000	9 000	10 100	8 200	8 000	7 900
2	10 200	10 000	9 900	10 200	11 500	11 900	8 400	9 100	10 000	9 100	8 700	8 500
3	8 800	8 700	8 600	9 800	10 000	10 000	10 000	9 700	9 600	9 400	9 600	9 600
4	10 500	9 100	8 600	8 500	9 500	9 900	8 600	9 800	10 200	9 300	7 500	6 900
5	10 200	10 100	10 500	8 800	9 600	9 900	10 100	9 800	9 700	10 400	10 200	10 200
6	10 600	10 000	9 800	10 300	10 000	10 000	10 400	10 400	10 400	10 000	9 900	9 900
7	10 300	9 900	9 700	9 600	9 900	10 000	8 700	9 500	9 700	10 900	10 000	9 700
8	9 800	9 600	9 500	10 200	10 900	11 000	7 600	9 600	10 300	7 800	9 900	10 600
Column No. 4, Test No. 8.												
1	9 000	9 800	10 000	10 000	10 700	11 000	8 200	10 000	10 800	9 000	9 200	9 300
2	8 700	9 300	9 500	10 100	10 800	11 100	10 100	11 600	12 100	8 600	9 400	9 700
3	9 300	9 800	9 900	10 800	11 200	11 400	10 000	10 200	10 200	7 400	8 400	8 700
4	9 500	9 700	9 800	11 100	10 200	10 000	9 700	11 100	11 600	7 100	8 200	8 500
5	10 200	10 500	10 600	11 900	12 400	12 600	8 400	9 400	9 800	7 700	8 800	9 200
6	11 600	12 000	12 100	12 400	13 600	13 900	9 500	10 800	11 100	7 400	9 500	10 200
7	12 800	12 500	12 400	13 400	14 200	14 600	7 100	8 300	8 700	6 400	6 700	6 700
8	8 600	11 600	12 700	13 000	11 300	10 700	6 600	8 500	9 200	6 200	7 800	8 400
Column No. 5, Test No. 9.												
½	14 000	11 700	10 800	12 600	11 700	11 400	7 400	8 100	8 400	7 400	7 900	8 000
1	11 300	12 000	12 300	11 100	10 000	9 600	6 700	8 500	9 200	5 100	6 800	7 500
1½	11 300	11 100	11 100	11 100	10 800	10 700	5 700	6 200	6 300	6 600	6 900	7 100
2	13 100	12 600	12 400	12 900	12 400	12 200	8 000	8 000	8 000	7 800	8 200	8 300
2½	12 700	12 000	11 700	13 100	11 500	11 100	9 300	9 200	9 200	8 700	8 400	8 300
3	11 800	12 000	12 200	11 400	12 200	12 400	8 200	8 400	8 400	6 300	8 000	8 600
3½	13 500	10 500	9 300	11 600	10 700	10 400	8 200	8 200	8 200	8 400	8 300	8 200
4	14 300	13 100	12 700	14 300	12 300	11 500	7 300	7 400	7 500	7 400	8 700	9 200
4½	11 600	11 600	11 500	12 300	12 300	12 300	9 000	9 000	9 000	8 700	8 700	7 800
5	12 600	12 400	12 300	11 700	10 400	10 000	8 300	8 400	8 400	7 400	8 500	8 100
5½	13 700	12 100	11 500	13 500	12 100	11 400	8 300	8 300	8 300	8 400	8 400	7 600
6	12 600	12 700	12 700	13 500	12 500	12 200	7 600	7 600	7 600	7 200	7 000	7 100
6½	13 100	11 900	11 500	11 400	11 400	11 500	8 400	8 400	8 400	10 500	9 000	8 500
7	11 100	11 300	11 500	11 200	12 300	12 600	7 900	8 000	8 000	6 500	8 100	8 600
7½	10 600	10 100	9 900	11 000	10 300	9 900	8 000	8 200	8 200	5 900	7 900	8 700
8	10 300	12 300	12 800	11 300	12 300	12 400	8 500	8 600	8 600	10 800	11 800	12 200

TABLE 3—(*Continued*).

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Column No. 4*a*, Test No. 10.												
½	8 100	9 500	10 200	8 400	8 100	8 000	11 700	10 800	10 500	9 900	11 200	11 800
1	8 300	10 200	10 800	6 500	7 200	7 500	12 100	10 900	10 400	11 100	12 500	13 100
1½	8 400	9 400	9 700	7 400	8 200	8 400	12 100	11 400	11 100	11 000	11 400	11 600
2	8 700	8 800	9 000	6 200	7 200	7 500	10 800	10 500	9 700	9 300	11 700	12 600
2½	8 300	8 800	9 000	7 300	6 900	6 800	11 800	11 500	12 100	10 900	10 400	10 300
3	8 700	9 700	9 900	6 700	6 500	6 400	13 300	11 100	10 400	10 400	11 100	11 400
3½	8 300	8 000	8 000	7 300	7 300	7 400	13 600	11 600	11 100	13 000	12 400	12 400
4	8 200	9 700	10 100	6 800	6 900	6 800	14 300	13 500	12 700	13 100	11 100	10 500
4½	7 500	8 600	9 100	6 300	7 200	7 500	12 300	11 100	10 800	10 300	15 200	9 400
5	7 500	8 400	8 800	7 000	8 100	8 500	13 000	13 300	13 500	9 000	11 400	12 300
5½	6 300	7 200	7 500	5 900	6 100	6 200	12 700	12 300	12 100	12 300	10 700	10 100
6	6 800	7 800	8 200	5 360	6 800	7 200	14 200	14 200	14 200	11 000	11 200	11 300
6½	6 600	7 800	8 200	7 100	7 100	7 100	15 400	13 700	12 900	13 500	11 500	11 100
7	6 700	7 100	7 200	4 900	6 100	6 700	11 900	13 600	14 600	11 100	12 100	12 400
7½	6 100	7 600	8 300	4 800	8 100	9 400	12 300	12 400	12 400	9 300	16 200	11 100
8	6 000	8 200	9 000	8 400	6 800	6 200	10 700	12 200	12 800	10 000	16 300	11 100
Column No. 2*a*, Test No. 11.												
½	11 600	10 900	10 400	11 400	10 500	10 000	......	......	......	......	......	......
1	7 300	9 300	10 000	7 300	9 700	10 400	8 400	8 800	9 000	10 200	10 200	10 200
1½	11 600	9 900	9 600	11 600	10 900	10 000	8 500	10 000	10 600	11 400	12 000	12 200
2	8 900	8 700	8 600	10 500	10 300	10 000	10 200	9 000	8 600	11 500	11 200	11 000
2½	11 200	9 700	9 200	10 900	10 300	10 000	9 900	10 000	10 200	11 700	10 300	9 900
3	9 500	10 500	10 700	12 200	10 800	10 000	11 800	10 200	10 800	11 400	10 600	10 300
3½	10 500	11 000	11 000	12 100	10 600	10 000	10 400	8 600	8 100	9 700	9 600	9 400
4	9 800	9 500	9 400	10 600	9 900	9 600	9 800	9 400	9 200	10 400	10 600	10 500
4½	10 300	9 600	9 300	11 200	10 200	10 000	8 700	9 600	10 400	10 400	10 000	10 000
5	9 600	9 800	9 900	10 400	10 800	11 000	10 300	9 200	8 800	10 400	10 200	10 200
5½	12 200	11 600	11 200	12 000	11 100	10 700	12 500	10 400	10 000	12 200	11 900	11 700
6	10 100	9 700	9 700	9 900	9 700	9 700	9 100	10 400	10 900	10 300	10 700	10 700
6½	11 300	11 000	10 800	10 600	9 900	9 700	10 800	9 900	9 700	11 000	11 000	10 000
7	9 300	9 600	9 709	9 100	9 300	9 200	9 800	9 800	9 800	9 800	10 800	10 000
7½	10 200	10 900	11 400	9 100	9 400	9 500	9 300	9 600	9 800	9 900	10 700	10 800
8	9 900	9 800	9 700	11 100	10 000	9 600	8 800	10 300	10 900	10 800	10 700	10 800
Column No. 2*a*, Test No. 12.												
½	10 600	10 000	9 800	10 400	10 600	10 600	11 500	10 600	10 000	13 200	12 600	12 300
1	8 400	8 400	8 500	9 700	10 800	11 200	9 800	10 700	10 900	10 000	11 300	11 700
1½	8 200	8 000	7 900	9 200	10 600	10 900	9 200	9 660	9 700	8 800	10 000	10 200
2	9 200	9 800	10 000	9 900	10 900	11 300	11 400	10 000	9 400	12 200	11 400	11 000
2½	10 800	9 900	9 600	10 800	9 600	9 100	10 900	10 900	10 900	11 400	11 600	11 600
3	9 100	10 800	11 200	11 000	10 600	10 400	10 600	11 200	11 300	10 400	10 200	10 100
3½	10 700	10 900	10 800	12 000	10 600	10 000	10 600	10 900	10 900	10 200	10 600	10 600
4	10 000	8 800	8 400	11 400	9 400	9 400	8 900	10 000	10 300	8 800	10 300	10 960
4½	10 600	10 000	9 900	10 200	10 000	9 900	10 000	10 100	10 200	8 606	9 500	9 800
5	10 100	10 300	10 400	11 300	11 000	11 000	11 800	10 200	9 600	10 200	9 700	9 700
5½	10 900	11 300	11 300	11 400	10 600	10 300	13 000	11 000	10 400	11 000	9 500	9 000
6	11 500	10 800	10 500	9 500	9 000	9 000	9 500	9 900	10 000	9 700	9 500	9 500
6½	11 700	9 800	9 200	12 700	10 300	9 600	10 000	10 000	10 000	9 600	10 300	10 600
7	9 900	11 300	11 600	9 900	11 100	10 600	10 500	9 800	9 300	9 600	10 000	10 000
7½	8 000	9 700	10 300	15 490	15 400	15 400	7 800	10 000	10 600	6 800	8 500	9 000
8	10 000	9 800	9 900	11 700	10 100	9 900	8 800	10 400	11 100	9 400	9 400	9 400

TABLE 3—(*Continued*).

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
	Column No. 2a, Test No. 13.											
1	4 500	7 000	8 600	7 000	8 600	9 100	9 800	10 800	11 100	10 300	12 300	13 000
2	7 600	9 000	9 500	10 300	10 400	10 600	10 900	10 100	9 700	12 800	12 600	12 600
3	7 000	8 600	9 100	9 400	10 100	10 300	11 300	11 300	11 300	11 600	12 600	13 000
4	8 100	9 100	9 500	10 400	10 800	10 900	9 300	10 400	10 700	9 600	11 600	12 500
5	8 200	9 500	10 000	10 000	10 200	10 200	10 900	8 900	8 100	10 600	10 700	10 800
6	10 000	10 400	10 600	10 500	10 000	10 000	8 500	9 600	10 000	9 600	10 400	10 800
7	9 800	9 800	9 900	10 200	10 300	10 300	9 500	8 800	8 700	9 900	10 000	10 100
8	8 000	7 800	7 700	12 300	10 300	9 200	7 700	8 700	9 100	8 700	10 000	10 500

Table 4 gives a number of the most marked deviations from average stress. The excess of the maximum fiber stress is given as a percentage of the average stress.

TABLE 4.—Maximum Observed Fiber Stresses in Flange Members of Columns.

Column Number.	Test Number.	Features of Lacing.	Method of Loading.	Percentage of excess of maximum fiber stress over average stress. Highest five values.
1	1	$1\frac{1}{4}$ by $\frac{7}{16}$ in., bolted.....	Central.	42, 41, 39, 31, 23
1	2	"	Slightly eccentric.	68, 64, 50, 49, 35
1	3	"	"	50, 41, 35, 32, 19
1	4	1 by $\frac{1}{4}$ in., bolted.......	Central.	35, 29, 28, 27, 26
1	5	1 by $\frac{1}{4}$ in	"	67, 55, 49, 29, 27
2	6	....	"	31, 23, 23, 21, 17
3	7	....	"	20, 17, 12, 11, 9
4	8	....	"	41. 29, 24, 22, 19
5	9	....	"	43, 43, 38, 35, 35
4a	10	....	"	53, 49, 42, 40, 37
2a	11	....	"	21, 16, 13, 12, 11
2a	12	....	Oblique arm, 1 in. (Fig. 5).	42, 42, 24, 23, 20
2a	13	....	Oblique arm, 2 in. (Fig. 5).	35, 34, 22, 21, 21

In most cases the maximum stress was in the outer fiber of the channel; sometimes very high stresses were found in the inner fiber. Generally, the stress in the opposite channel was correspondingly less.

STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1

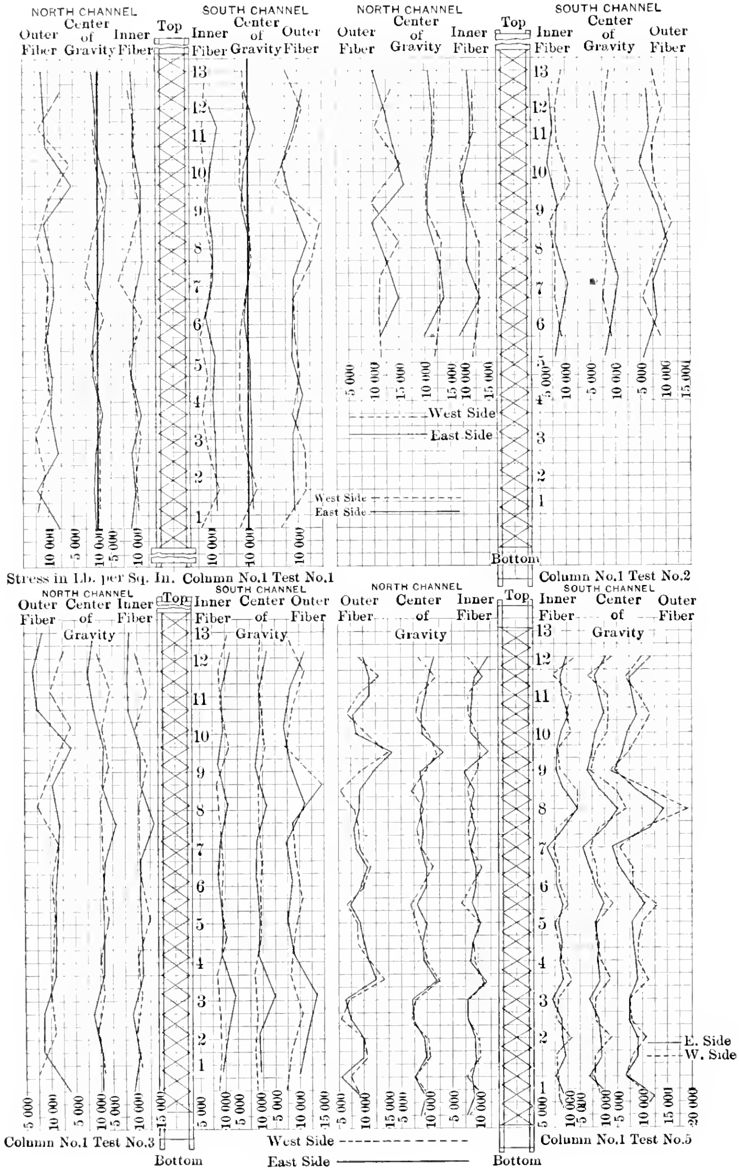


Fig. 6.

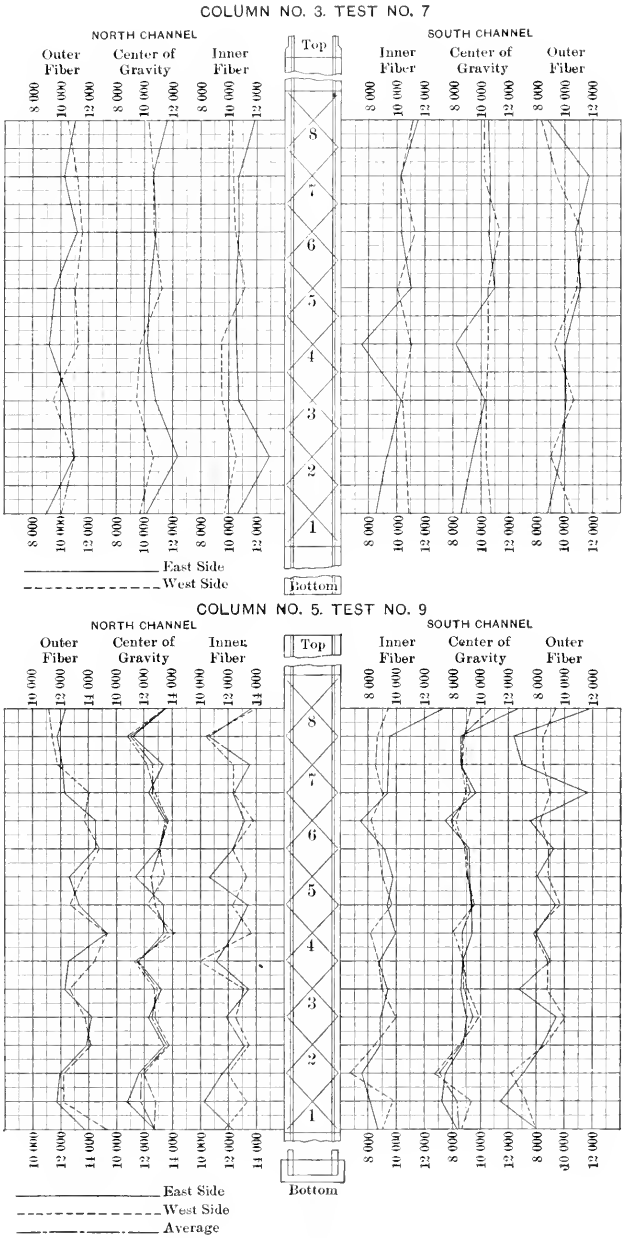

COLUMN NO. 3. TEST NO. 7
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
8 000
10 000
12 000
East Side
West Side
Bottom
COLUMN NO. 5. TEST NO. 9
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
10 000
12 000
14 000
8 000
East Side
West Side
Average
Bottom

FIG. 7.

Stress in Lattice Bars.—Table 5 gives the results of tests to determine the average stress in the various lattice bars of the columns. Tests 14 and 15 were tests on the lattice bars only. The distribution of stress over the cross-section of the bar is discussed in another place. The average stresses in the lattice bars are computed from the observed deformations, using a modulus of elasticity of 28 000 000 lb. per sq. in. for the steel column and 26 000 000 lb. per sq. in. for the wrought-iron columns. As might be expected, from the irregular variation of stress along the flange members of the columns, the stress in the lattice bars was found to vary greatly.

Table 6 gives the largest stresses observed and the corresponding transverse shear. The transverse shear given in this table is that which would cause a stress in the lattice bars equal to the maximum stress observed in any lattice bar, and was computed by doubling the transverse component of the maximum load observed on a lattice bar. In the case of obliquely loaded columns, the transverse component of the load was computed on the assumption that the load was applied

TABLE 5.—TOTAL STRESS, IN POUNDS, ON LATTICE BARS UNDER LOAD ON COLUMNS OF 10 000 LB. PER SQ. IN.

COLUMN NO. 1.

Lattice bar.	Test No. 5.		Test No. 14.		Test No. 15.	
	Front.	Back.	Front.	Back.	Front.	Back.
1	400c	400c	300t	400t	......	......
2	100t	600t	1 600t	400c	......	......
3	0	600c	800c	400t	......	......
4	100t	400t	300t	100c	......	......
5	100c	100c	......	......	......	......
6	200t	300t	......	......	......	......
7	100c	200c	......	......	......	......
8	200t	200t	......	......	......	......
9	0	300c	......	......	......	......
10	0	200t	......	......	......	......
11	100c	300c	......	......	......	......
12	0	100t	......	......	......	......
13	100c	200c	800c	0	......	......
14	100t	200t	900t	900t	2 500t	1 600c
15	0	300c	200c	1 000t	2 100c	......
16	0	400t	800t	0	1 600t	200c
17	200c	200c	500c	300t	1 900c	......
18	200t	100t	700t	1 000t	700t	1 100t
19	500c	200c	500c	200t	1 500c	......
20	200t	300t	800c	200t	1 400t	700t
21	0	600c	800c	600t	1 500c	0
22	100c	2 000t	0	200t	700t	300t
23	200t	900c	300c	800c	1 300c	......
24	200c	800t	200t	800t	600t	400c
25	200t	800c	0	0	2 700c	......
26	200c	700t	400t	300c	400t	400c

TABLE 5—(*Continued*).

COLUMN NO. 2*a*, TEST NO. 11.

Lattice bar.	Front Side.		Back Side.	
	Under.	Over.	Under.	Over.
1	700*c*	700*c*	1 300*c*	100*c*
2	100*c*	100*t*	200*t*	200*t*
3	3 000*t*	0	1 000*t*	0
4	100*c*	100*c*	800*c*	200*c*
5	3 000*t*	0	800*c*	300*c*
6	200*t*	200*c*	0	300*c*
7	3 700*t*	0	200*t*	200*c*
8	0	800*c*	800*c*	300*c*

COLUMN NO. 2*a*, TEST NO. 12.

1	1 500*c*	2 100*t*	200*c*	200*c*
2	200*c*	400*t*	2 300*t*	200*c*
3	800*t*	200*t*	2 750*t*	400*c*
4	700*c*	500*t*	2 750*t*	300*c*
5	1 600*t*	200*t*	2 750*t*	300*c*
6	0	0	2 800*t*	800*c*
7	1 400*t*	800*c*	4 100*t*	300*c*
8	100*t*	800*c*	300*t*	800*c*

COLUMN NO. 2*a*, TEST NO. 13.

1	1 000*c*	700*t*	800*t*	500*c*
2	500*c*	700*t*	2 650*t*	400*c*
3	700*c*	900*t*	2 750*t*	300*c*
4	500*c*	1 000*t*	2 100*t*	200*t*
5	800*c*	600*t*	3 150*t*	1 000*c*
6	1 000*c*	700*t*	4 100*t*	700*c*
7	700*c*	900*t*	1 500*t*	400*c*
8	1 600*c*	200*c*	2 000*t*	1 400*c*

through the center of the bearing blocks. This transverse component was then subtracted from the amount of shear which had been calculated from the deformation of the lattice bars as before noted, and the remainder has been tabulated under the heading "Transverse shear in column due to nominal central load."

Tests to Failure.—After the wrought-iron bridge posts had been tested for stress distribution under working loads, they were loaded to failure. Deformations were measured in the flange members of that part of the column on which the previous test had given the heaviest stress. Table 7 gives the result of the tests to failure. For all the tests of wrought-iron bridge posts, whether loaded centrally or eccentrically, the failures were very gradual. Final failure occurred near the middle or at the end. In the former case, high stresses in one channel had been shown by the deformation measurements at working

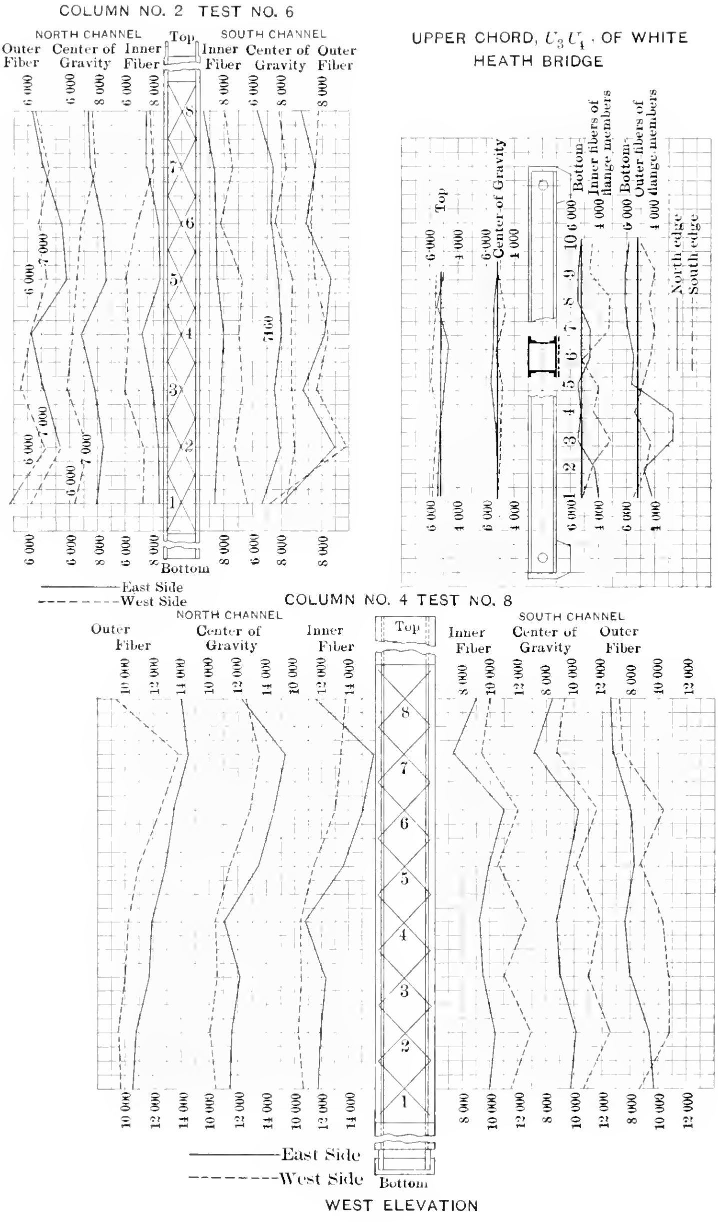

COLUMN NO. 2 TEST NO. 6
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
Bottom
East Side
West Side
UPPER CHORD, $U_3 U_4$, OF WHITE HEATH BRIDGE
Center of Gravity
Inner fibers of flange members
Outer fibers of flange members
North edge
South edge
COLUMN NO. 4 TEST NO. 8
WEST ELEVATION

Fig. 8.

TABLE 6.—MAXIMUM OBSERVED AVERAGE STRESS IN LATTICE BARS OF COLUMNS NOS. 1 AND 2*a*.

Column number	1	1	1	2*a*	2*a*	2*a*
Test number	5	14	15	11	12	13
Method of loading	Central.	Central.	Oblique arm, 4 in. (Fig. 5).	Central.	Oblique arm, 1 in. (Fig. 5).	Oblique arm, 2 in. (Fig. 5).
Total load, in pounds	187 600	187 600	187 600	176 400	176 400	176 400
Lacing	Single, 63° 30′ with axis, 1 by ¼ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.
Maximum observed average stress in lattice bar, in pounds per square inch	8 200	3 000	4 900	3 900	4 350	4 350
Corresponding total stress on bar, in pounds	2 000	1 600	2 700	3 700	4 100	4 100
Corresponding transverse shear in column, in pounds	3 700	2 900	4 8 0	5 200	5 800	5 800
Shear due to known eccentricity of load, in pounds	0	0	3 000	0	1 000	2 000
Transverse shear in column due to nominal central load, in pounds	3 700	2 900	1 8 0	5 200	4 800	3 800
Ratio of shear to compression load	0.020	0.016	0.009	0.029	0.027	0.021
Next highest observed values of ratio of shear to compressive load	0.009 0.008 0.008 0.007	0.010 0.010 0.009 0.009	0.009 0.607 0.006 0.006	0.024 0.023 0.010 0.008	0.019 0.018 0.018 0.018	0.016 0.014 0.014 0.011

loads. In the latter case, a bending in one channel at working loads was noted by the instruments at the panel nearest the end of the column. In two of the three columns in which failure took place in the end of the column, as the instruments did not show over-stress in the laced portion of the column, the injured ends were removed, new end connections were put on, and the columns were retested as Nos. 2*a* and 4*a*.

TABLE 7.—RESULTS OF TESTS TO FAILURE.

Column Number..	2	3	4	5	2*a*	4*a*
Test number......	6	7	8	9	13	10
Method of loading	Central.	Central.	Central.	Central.	Oblique arm. 2 in. (Fig. 5).	Central.
Load at failure. in pounds..........	466 000	480 000	452 000	492 000	475 000	475 000
Average stress at failure, in pounds per square inch..	26 400	27 200	25 700	27 900	26 900	26 900
Average stress at failure in tests of short pieces of flange members, in pounds per inch............	36 800	36 800	36 800	36 800	36 800	36 800
Percentage of excess of ultimate strength of short pieces over column strength.	39.5	35.5	43.5	32.0	37.0	37.0
Load at first sign of yielding, in pounds..........	317 000	264 000	256 000	264 000	......	......
Average stress at first sign of yielding, in pounds per square inch.	18 000	15 000	14 500	15 000	......	..... .
Method of failure, and remarks....	End failed.	End failed.	End failed.	Bowed at middle in plane perpendicular to lacing.	Buckled near bottom in plane parallel to lacing.	Bowed at middle in plane perpendicular to lacing.

Failure by Buckling of Lattice Bars.—In one of the tests of Column No. 1, the lattice bars failed by buckling suddenly and without warning. As tested, the column was fitted with the light lattice bars (1 by $\frac{1}{4}$ in.) riveted in place. The test had in view the trial for stress distribution under a slight obliquity, which was not carefully determined. No measuring instruments were in place. A preliminary load was being applied. When the load reached 150 000 lb. (8 060 lb. per sq. in. of cross-section), the alternate lattice bars in the upper half of the column buckled. A failure of this kind was quite unexpected at such a low load. Although an observer was watching the column, the failure was so sudden that he was unable to follow the movement of the parts. In this respect it was quite in contrast to the

failure of the other columns. The machine was at once stopped. Little damage was done to the column, except to the lacing bars. The webs were easily straightened, new lacing bars put on, and the column was used in another test.

Tests to destruction under compression had previously been made on lattice bars like those used in this column, and the results, in the absence of other data, may be useful in estimating the load carried by the lattice bars at failure. Under conditions of loading similar to the conditions found in column lattice bars, these sample bars failed under an average load of 2 100 lb. Assuming that the bar in this column which first failed was carrying 2 100 lb. when failure occurred, the transverse shear in the column may be computed. The following is a tabulated statement of the results of this test:

Column.	Compressive load, in pounds.	Lacing.	Manner of loading.	Probable maximum load on lattice bar, in pounds.	Corresponding shear in column, in pounds.	Ratio of transverse shear to compression load.
No. 1.......	150 000	Single 63° 30′ 1 by ¼, riveted.	Very slight obliquity.	2 100	3 760	0.0251

The column, now riveted up with the heavier lacing bars ($1\frac{1}{4}$ by $\frac{7}{16}$ in.), was loaded obliquely, as shown in Fig. 5, with 300 000 lb. (15 900 lb. per sq. in. of cross-section), and the stresses in the lacing bars were measured. No sign of failure was apparent, and, from the data given in a succeeding paragraph, it appears that under this load the lattice bars must have been stressed to about three-quarters of their ultimate strength in compression.

Cross-Bending Tests of Columns.—Cross-bending tests were made on one of the wrought-iron columns and on Column No. 1. The tests were made in an Olsen 200 000-lb. testing machine fitted for testing beams 20 ft. long. The columns were supported at the ends and loaded at the center with a light transverse load. The column was placed first with the plane of the lacing perpendicular to the load, and then with the plane of the latticing parallel to the load. The lattice bars used in the tests of Column No. 1 were $1\frac{1}{4}$ by $\frac{7}{16}$ in. in cross-section; in one test they were bolted in place and in another they were riveted. The deflection at various points along the beam was measured

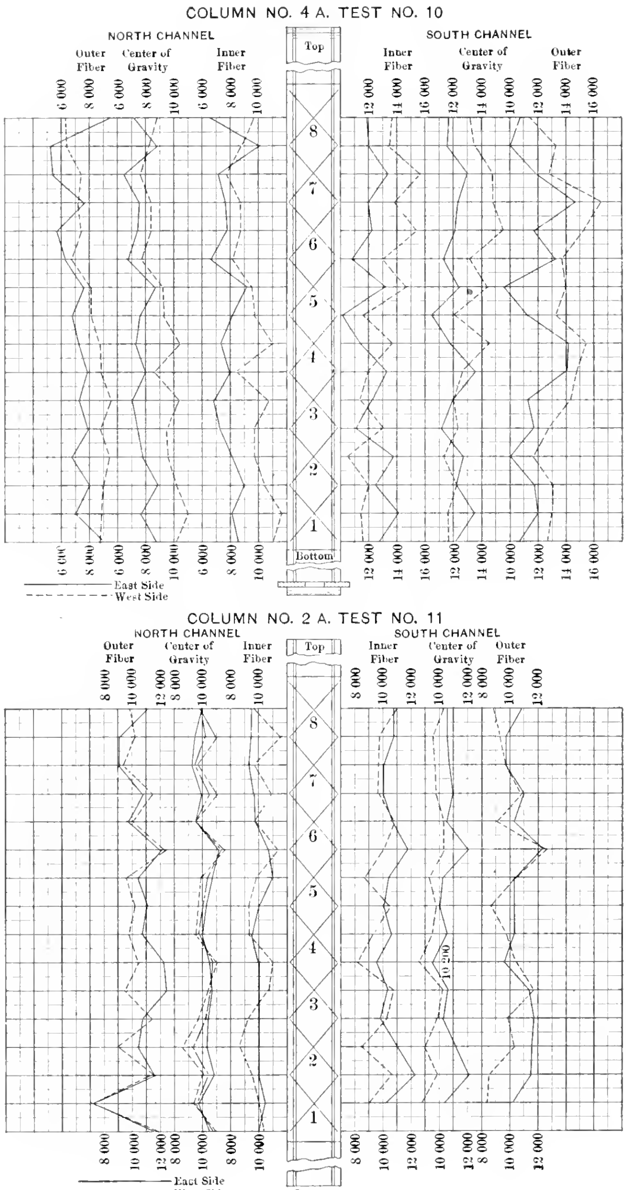

COLUMN NO. 4 A. TEST NO. 10
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Top
Bottom
6 000
8 000
10 000
12 000
14 000
16 000
East Side
West Side
COLUMN NO. 2 A. TEST NO. 11
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Top
Bottom
8 000
10 000
12 000
10 200
East Side
West Side

FIG. 9.

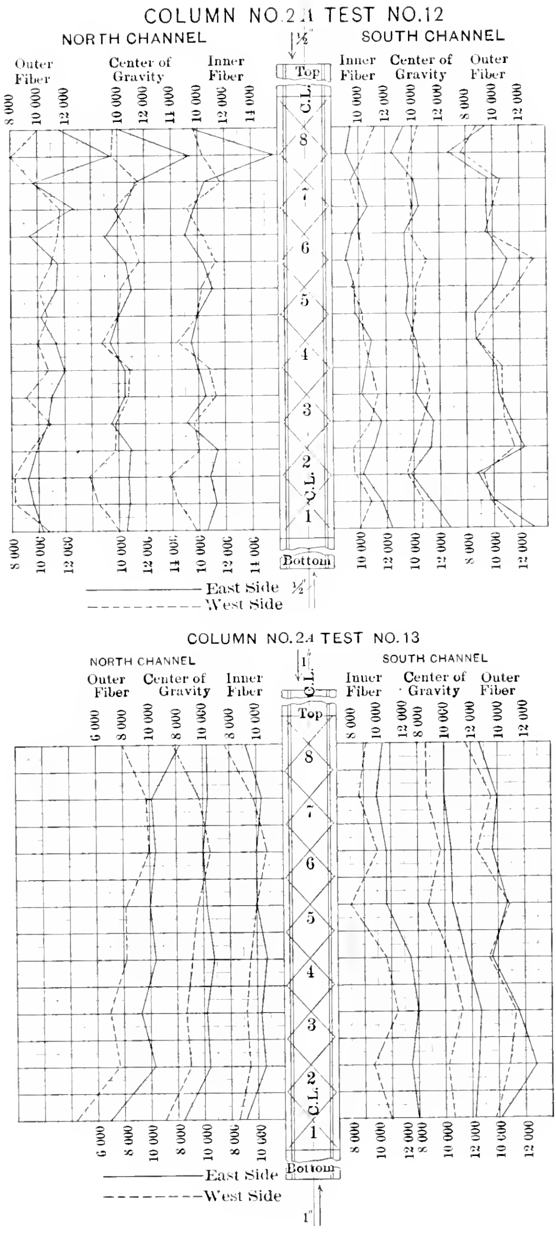

COLUMN NO.2A TEST NO.12
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
C.L.
Bottom
East Side
West Side
½"
COLUMN NO.2A TEST NO.13
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
C.L.
Bottom
East Side
West Side
1"

FIG. 10.

with Ames test gauges, and the actual curve assumed by the column under transverse load was thus determined. The theoretical elastic curve was computed from the common theory of flexure, not counting the lattice bars in the calculation of the moment of inertia. Fig. 2, Plate XI, shows the deflectometers and extensometers on Column No. 1 under the cross-bending test. Fig. 14 shows the deflection curves given by the column under transverse load and also the computed elastic curves.

It will be noted that when tested with the lacing vertical, Column No. 1 shows much greater deflection than that computed from the usual beam formula, while the stiffer wrought-iron column shows a much closer agreement with the curve, the heavy lacing apparently adding stiffness.

Field Tests of Columns.

Description of Bridge.—The bridge on which the field tests were made is an eight-panel single-track Pratt truss which spans the Sangamon River near White Heath, Illinois. The bridge is on the line of the Illinois Central Railroad between Champaign and Clinton, Illinois. The span measured 158 ft. 6 in. Fig. 12 gives a diagram of the bridge and Fig. 13 is from a photograph of the bridge.

Loads.—The test load applied to the bridge consisted of a mogul locomotive and tender followed by a loaded coal car and a caboose. Fig. 11 shows the test train, with dimensions and weights. This train was furnished through the courtesy of the railroad.

Members Investigated.—The members studied for stress distribution were Posts U_2L_2 South, U_3L_3 South, U_3L_3 North, and upper chord U_3U_4 South. The bridge diagram, Fig. 11, shows the location of the members. The posts were made up of two steel channels, double laced, while the top chord was made up of two built-up channels with a cover-plate on top and lacing across the bottom. The cross-section of the members tested is shown in Fig. 1. Fig. 12 shows the post and the chord.

Measurement of Deformation.—Ames test gauges were used as extensometers, and the method of attachment was the same as in the laboratory tests of columns. The method of reduction of instrument readings to stresses at the extreme fibers of members was also the same.

Routine of Tests.—As in the laboratory tests the stress distribution in the channels and the lattice bars was studied. The method of

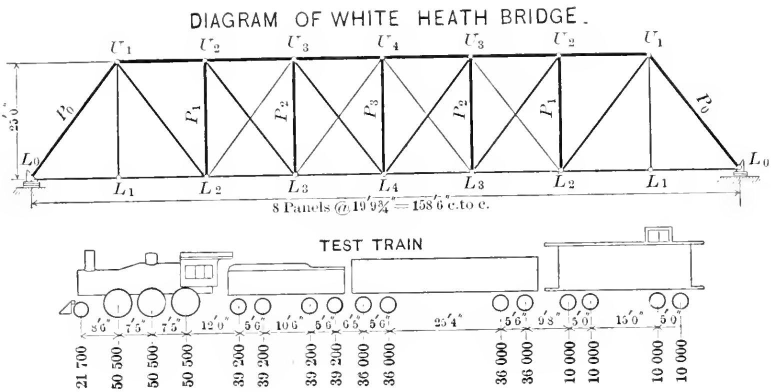


DIAGRAM OF WHITE HEATH BRIDGE.
8 Panels @ 19′ 9¾″ = 158′ 6″ c. to c.
TEST TRAIN
21 700
50 500
50 500
50 500
39 200
39 200
39 200
39 200
36 000
36 000
36 000
36 000
10 000
10 000
10 000
10 000

FIG. 11.

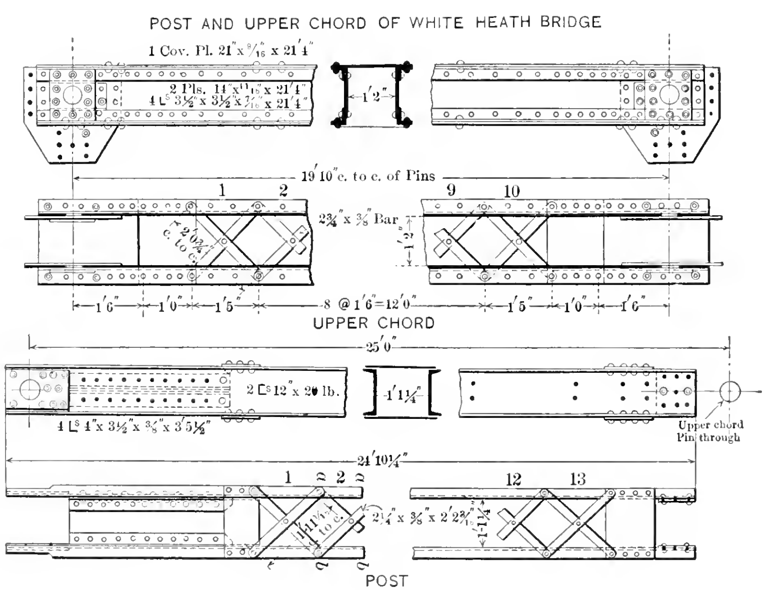

POST AND UPPER CHORD OF WHITE HEATH BRIDGE
1 Cov. Pl. 21″ x 9/16″ x 21′ 4″
2 Pls. 14″ x 11/16″ x 21′ 4″
19′ 10″ c. to c. of Pins
2¾″ x ⅜″ Bar
UPPER CHORD
25′ 0″
2 [s 12″ x 20 lb.
4 Ls 4″ x 3½″ x ⅜″ x 3′ 5½″
Upper chord Pin through
24′ 10¼″
POST

FIG. 12.

procedure was as follows. Instruments were placed on some portion of the column to measure the deformation over a short gauge length, and a reading was taken. The test train was then run upon the bridge to a given position (one approximating the maximum load on the member under test), and the instruments were read again. The train was then run off the bridge, and the instruments were again read. This procedure was repeated several times, at least three applications of the load being made and frequently several more. The instruments were then moved to another part of the column, and that part was tested. Observations were made on both flange members and lattice bars. The tests covered a period of eight days.

TABLE 8.—Stresses in Posts of White Heath Bridge.

Post U_3L_3 South.

	North Channel.						South Channel.					
	East side.			West side.			East side.			West side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
1*	5 000	5 300	5 400	2 600	2 700	2 700	4 900	5 000	5 000	2 300	2 500	2 600
2	3 400	3 500	3 500	1 900	1 600	1 500	3 300	3 400	3 400	1 600	1 800	1 800
3	3 500	3 800	4 000	1 800	1 900	2 000	2 600	2 900	3 100	2 600	2 200	2 100
4	3 700	3 400	3 300	2 800	3 000	3 000	1 600	3 100	3 600	2 400	2 800	2 900
5	3 500	3 900	4 000	2 900	2 900	2 900	2 800	3 000	3 100	2 200	2 600	2 800
6	3 600	3 600	3 600	3 400	3 800	3 900	3 400	3 400	3 400	4 100	3 400	3 200
7	2 600	3 200	3 400	3 300	3 100	3 000	2 400	3 000	3 300	2 600	2 700	2 800
8	3 300	3 200	3 200	3 900	3 600	3 500	3 600	3 400	3 300	3 700	3 900	3 900
9	3 200	3 100	3 000	3 600	3 300	3 200	1 700	2 500	2 800	2 900	3 300	3 400
10	2 800	2 700	2 700	3 100	3 300	3 300	2 700	2 400	2 300	3 300	3 700	3 900
11	3 300	2 900	2 800	4 500	3 400	3 100	2 900	2 000	1 700	3 500	3 900	4 100
12	1 400	2 000	2 300	3 100	3 200	3 200	1 800	1 800	1 800	4 800	4 700	4 600
13	1 300	1 500	1 500	2 800	2 700	2 600	2 600	2 900	3 100	5 200	4 900	4 800

Post U_3L_3 North.

Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
1	4 900	4 900	4 900	2 600	2 500	2 500	4 800	4 900	4 900	2 100	2 700	2 800
1½	4 800	4 700	4 700	2 900	2 800	2 700	3 100	4 900	5 700	900	1 500	1 700
2	4 800	4 400	4 300	2 600	2 500	2 500	4 900	4 700	4 600	2 700	1 700	1 300
2½	4 000	3 900	3 900	2 700	2 800	2 900	4 300	3 700	3 500	2 300	1 800	1 600
3	3 900	4 200	4 300	2 800	2 900	2 900	......	......	......	......	......	......
3½	3 900	3 900	4 000	2 400	2 500	2 500	3 200	3 800	4 100	2 400	2 500	2 500
4	3 400	4 600	5 000	2 700	2 700	2 700	3 900	3 700	3 700	3 500	3 200	3 100
4½	3 800	3 400	3 200	2 500	2 300	2 200	3 500	3 500	3 400	3 100	3 000	3 000
5	3 900	3 700	3 600	3 700	3 700	3 700	3 100	3 600	3 800	1 400	2 100	2 400
5½	4 100	3 600	3 400	4 000	3 400	3 200	3 500	3 900	4 100	3 500	2 900	2 800
6	3 600	3 200	3 100	3 400	3 300	3 200	3 700	3 500	3 400	3 900	3 600	3 500
6½	3 300	3 300	3 300	3 600	3 500	3 500	4 500	4 700	4 800	2 000	2 400	2 600

* No explanation for the high values in Panel 1 has been found. Five determinations of stress were made, including the removal and re-attachment of instruments.

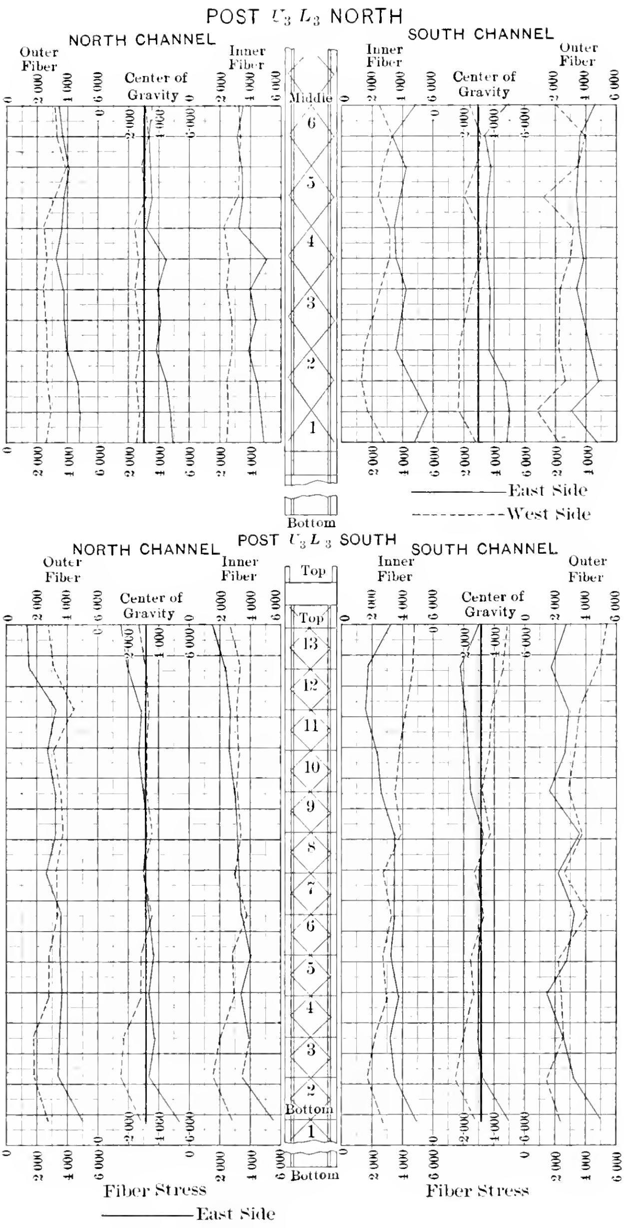

POST $U_3 L_3$ NORTH
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Inner Fiber
Center of Gravity
Middle
Bottom
East Side
West Side
POST $U_3 L_3$ SOUTH
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Inner Fiber
Center of Gravity
Top
Bottom
Fiber Stress
Fiber Stress
East Side
West Side

FIG. 13.

Results of Tests for Stress Distribution in Channels.—Table 8 gives the results of the tests to determine the stress distribution and variation in the channels of the bridge posts, and Table 9 gives those for the top chord. The stresses given are calculated from the observed deformations, using a modulus of elasticity of 30 000 000 lb. per sq. in. The conditions of measurement of deformation were much the same as in the laboratory tests. The stress noted is the average stress over a space of 4½ in. on either side of the point indicated.

TABLE 9.—Stresses in Upper Chord of White Heath Bridge.

Lower Side (Laced).					Upper Side (Cover-plate).				
	North Channel.		South Channel.						
Panel.	Outer fiber.	Inner fiber.	Outer fiber.	Inner fiber.	Distance from end, in inches.	North edge.	Over north web plate.	Over south web plate.	South edge.
1	4 300	3 900	5 000	5 500	33	5 500	6 000	5 200	5 700
2	4 900	4 300	4 500	4 800	57	....	....	5 200	5 500
3	2 800	5 500	3 400	4 500	81	5 200	5 400	5 900	5 300
4	2 800	5 100	4 200	5 300	105	5 000	6 100	5 700	6 000
5	5 700	5 400	3 900	4 800	129	4 800	5 900	5 900	5 900
6	5 500	4 800	5 100	4 200	153	5 400	5 700	5 500	5 600
7	5 900	4 700	3 400	3 600	177	5 000	5 000	5 700	5 600
8	6 200	5 300	3 300	4 400	....	....	....	....	....
9	6 000	5 300	4 700	3 800	....	....	....	....	....
10	5 900	5 100	4 400	4 900	....	....	....	....	....

TABLE 10.—Maximum Observed Fiber Stress in Flange Members of Columns in White Heath Bridge.

Column number	U_3L_3 South	U_3L_3 North	U_3U_4 South
Test number	F 1	F 2	F 3
Lacing	Double, 45°, riveted at crossing.	Double, 45°, riveted at crossing.	Cover-plate on top. Double, 45° on bottom.
Maximum observed compressive stress in an extreme fiber, in pounds per square inch	5 200	5 700	6 200
Percentage of excess maximum observed stress over average, highest five values.	73*	64	20
	63	48	19
	60	41	17
	57	31	17
	47	31	17

* No values from Panel 1 have been included in this table, as no explanation is known of the high stresses indicated in that panel.

Figs. 8 and 13 show graphically the stress distribution and variation. In these figures the full lines give the stresses at the west side (front), and the dotted lines the stresses at the east side (back).

In Table 10 are given a number of the highest observed fiber stresses. The excess of the maximum fiber stress is given as a per-

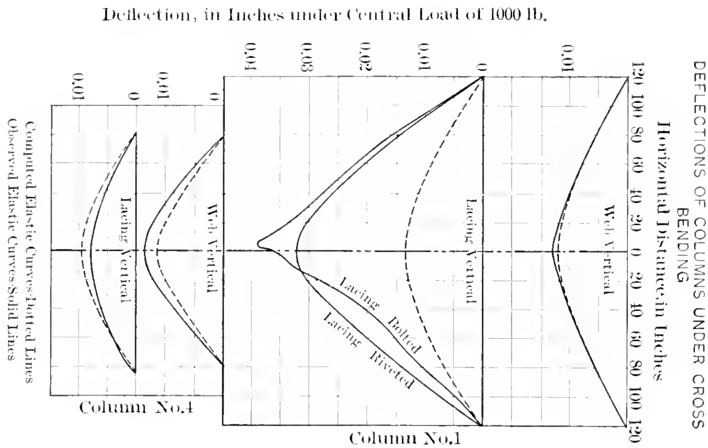


FIG. 14.

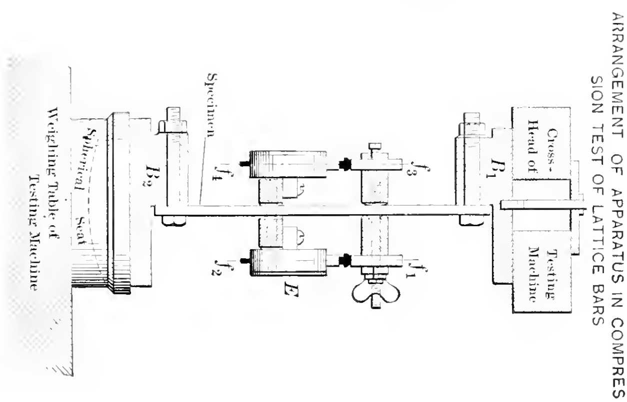


FIG. 15.

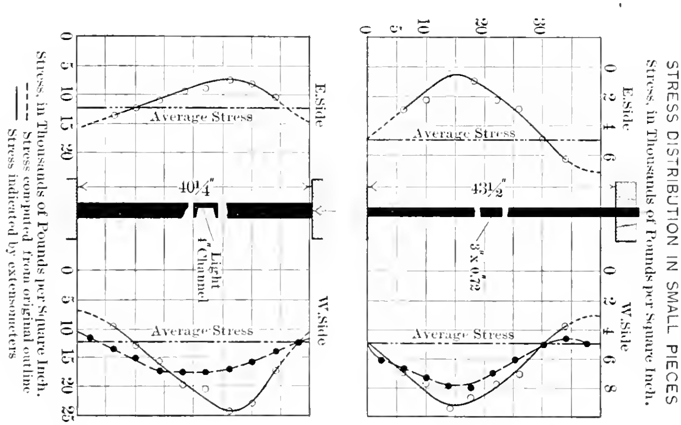


FIG. 16.

centage of the average stress. At most sections the maximum stress was in the outer fiber of the channel, but in some cases it was found at the inner fiber.

In the tests of the bridge posts an attempt was made to determine the stresses in a few of the lattice bars. These stresses were very small, and the precision of the extensometer was not sufficient to measure them with any great degree of accuracy. It should be noted that the lacing of the posts in this bridge was double, and the bars were riveted together at their intersection. In several cases it was found that a lattice bar under load bent in the shape of a very flat S curve, the point of attachment to another lacing bar, at the middle, being a point of inflection.

Special Tests on Bridge Columns.—Tests were made on the batten plates at the top of one of the posts, and under load a slight bending of the plates between channels was found. The bending took place in a horizontal plane.

In one post the change of stress was observed as the locomotive and train moved across the bridge. Extensometers were placed at *aa*, *bb* (Fig. 12), and on the floor beams, and the changes in reading were noted as the train moved across the bridge. In the inner channel of the post, tension was set up as the locomotive came opposite the post.

Tests of Lattice Bars, Small Columns and Column Material.

Compression Tests of Lattice Bars.—Many of the lattice bars in a column, as they transmit stress from one flange member of the column to the other, are under compression. To study the action of lattice bars under compression, a series of tests on single lattice bars was made. Fig. 15 shows the arrangement of the apparatus. The lattice bar was tightly bolted to the blocks, B_1 and B_2. The upper block, B_1, was fastened to the cross-head of a testing machine, and the block, B_2, was pressed against the weighing table of the testing machine. A spherical-seated bearing block was used, to insure an even bearing. Ames test gauges, E, mounted on suitable frames, were attached to the lattice bar over a short gauged length. From the readings of these gauges, the deformation of the extreme fiber of the bar was computed.

In this test of lattice bars, the load was applied with an eccentricity approaching that to be expected in a column for the lattice bars which

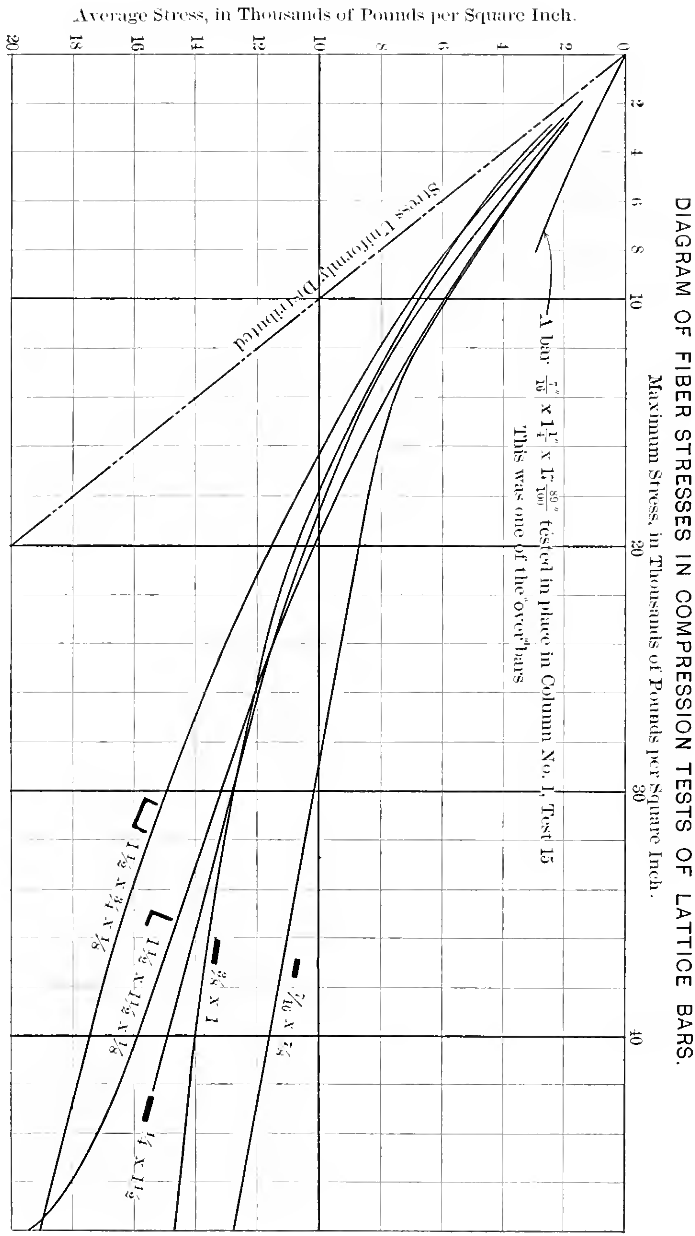


DIAGRAM OF FIBER STRESSES IN COMPRESSION TESTS OF LATTICE BARS.
Maximum Stress, in Thousands of Pounds per Square Inch.
Average Stress, in Thousands of Pounds per Square Inch.
Stress Uniformly Distributed
A bar 7/16" x 1 1/4" x 1' 89/100" tested in place in Column No. 1, Test 15
This was one of the "over" bars
1 1/2 x 3/4 x 1/8
1 1/2 x 1 1/2 x 1/8
3/8 x 1
7/16 x 7/8
1/4 x 1 1/2
0
2
4
6
8
10
12
14
16
18
20
30
40

Fig. 17.

are next to the flange member (here designated "under" bars). The lattice bars outside of these "under" bars are here designated "over" bars. The stress distribution across the section of the "over" bars, which are under compression, is probably more uneven than the stress distribution found in these tests. However, these tests give some idea of the relative behavior of lattice bars of various proportions, and of the large eccentricity of loading of all lattice bars.

Lattice bars of the following cross-sections were tested: Flat bars $1\frac{1}{2}$ by $\frac{1}{4}$ in., 1 by $\frac{3}{8}$ in., $\frac{7}{8}$ by $\frac{7}{16}$ in.; angles $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$ in.; channels $1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$ in. Several channel and angle lattice bars were tested with ends flattened and ribs turned inward, to minimize the eccentricity of loading. Bars of the following lengths between centers of rivet holes were tested: $8\frac{1}{2}$ in., $13\frac{1}{2}$ in., and 20 in. The rivet holes were $\frac{1}{2}$ in. in diameter. All bars were tested in a Philadelphia Machine Tool Company's 100 000-lb. testing machine, and loads and extensometer readings were taken to failure.

Observations were also made on the behavior of a lattice bar in a column under load, with a view to determine the distribution of stress over the section. Column No. 1 was loaded obliquely. The instruments were placed on an "over" bar which had been found to carry a high compressive stress, and readings were taken to determine the distribution of stress across the section.

When Column No. 1 was under cross-bending test, observations were made to determine the stresses transmitted by lattice bars and their distribution over the section of the bars. Extensometers were placed successively on most bars under compression on one-half of the column, and on some bars which were under tension. In both of these tests the bars were $1\frac{1}{4}$ by $\frac{7}{16}$ in., and were riveted.

Results of Tests of Lattice Bars.—The results of the tests of single lattice bars are given in Figs. 17 and 18, and in Tables 11 and 12. Fig. 17 shows the ratio of maximum to average stress in the bars $13\frac{1}{2}$ in. long between centers of rivet holes. It also gives the result of the test of stress distribution in a lattice bar of Column No. 1. Table 11 gives the stresses at failure of the various bars tested singly. The average stress on the various bars, which corresponds to a maximum fiber stress of 40 000 lb. per sq. in., as taken from these tests, has been noted and is given in Table 12. The results of the tests to failure are shown graphically in Fig. 18. The angle and channel bars

tested with flattened ends and ribs turned inward failed in the flattened part at loads no greater than the bars fastened with ribs turned outward.

TABLE 11.—COMPRESSION TESTS OF LATTICE BARS.

Average of two specimens.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	$\frac{l}{r}$	Average stress at failure, in pounds per square inch.	Stress at failure for steel of 40 000 lb. per sq. in. yield point, in pounds per square inch.*
$1\frac{1}{2}$ by $\frac{1}{4}$, flat	20	277	9 900	8 900
1 by $\frac{3}{8}$, flat	20	184	12 900	12 200
$\frac{7}{8}$ by $\frac{7}{16}$, flat	20	158	14 500	15 000
$1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$, channel	20	90	20 800	19 400
$1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$, angle	20	43	22 600	20 100
$1\frac{1}{2}$ by $\frac{1}{4}$, flat	$13\frac{1}{2}$	187	15 400	13 800
$\frac{7}{8}$ by $\frac{7}{16}$, flat	$13\frac{1}{2}$	107	16 300	16 800
1 by $\frac{3}{8}$, flat	$13\frac{1}{2}$	124	16 900	15 900
$1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$, channel	$13\frac{1}{2}$	61	Bolt sheared.	......
$1\frac{1}{2}$ by $\frac{1}{2}$ by $\frac{1}{8}$, angle	$13\frac{1}{2}$	29	Bolt sheared.	......
$1\frac{1}{2}$ by $\frac{1}{4}$, flat	$8\frac{1}{2}$	118	17 300	15 500
$\frac{7}{8}$ by $\frac{7}{16}$, flat	$8\frac{1}{2}$	67	18 100	18 700
1 by $\frac{3}{8}$, flat	$8\frac{1}{2}$	78	18 300	17 200
$1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$, channel	$8\frac{1}{2}$	38	Bolt sheared.	......
$1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$, angle	$8\frac{1}{2}$	18	Bolt sheared.	.....

* The values in this column were obtained by multiplying the observed stress at failure by $\frac{40\,000}{\text{yield point determined from tests}}$.

TABLE 12.—AVERAGE STRESS IN LATTICE BARS WHICH CORRESPONDS TO A MAXIMUM FIBER STRESS OF 40 000 LB. PER SQ. IN.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	Corresponding average stress, in pounds per square inch.
$\frac{7}{8}$ by $\frac{7}{16}$, flat	$13\frac{1}{2}$	11 600
1 by $\frac{3}{8}$, flat	$13\frac{1}{2}$	14 000
$1\frac{1}{2}$ by $\frac{1}{4}$, flat	$13\frac{1}{2}$	14 900
$1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$, angle	$13\frac{1}{2}$	15 900
$1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$, channel	$13\frac{1}{2}$	17 500

Table 13 gives the results of the test for stress distribution in the lattice bars of Column No. 1 as it was stressed in cross-bending.

Tests of Small Columns.—Tests of two small compression pieces were made in order to study the effect of slight bends and kinks in the column upon the distribution of stress. The deviation from a straight line, in these nominally straight pieces, was measured before the load was applied. The deformations on two opposite faces for a given load were measured. The extensometer was similar to that used on the

single lattice bar tests. The instrument was shifted from one position to another along the column. The columns were finally loaded to failure. One of the columns was a flat piece of steel, 3 by 0.72 in. in cross-section, and 40 in. long. It was held at the upper end by wedge grips in the cross-head of the machine and at its lower end rested on a spherical-seated block. The second compression piece was a 4-in. channel 40 in. long. The ends were planed square; the upper end bore on a flat compression plate in the iron head of the machine, and the lower end rested on a spherical-seated block.

TABLE 13.—Stress in Lattice Bars of Column No. 1 under Cross Bending.

Column tested as a beam centrally loaded over span of 19 ft. 8 in.; lattice bars $1\frac{1}{4}$ by $\frac{7}{16}$ in.; 17.89 in. center to center of rivet holes; rivets $\frac{1}{2}$ in. in diameter.

Bar.	Maximum fiber stress, from Ames dials.	Average stress, from Ewing extensometer.	Ratio, maximum to average.
"Over" Bars in Compression.			
14*E*	6 000	2 600	2.31
16*E*	8 300	2 000	4.15
18*E*	4 300	1 800	2.39
20*E*	7 700	1 400	5.50
22*E*	7 000	2 900	2.42
24*E*	8 900	2 200	4.04
			Average, 3.47
"Under" Bars.			
15*W*	4 300*c*	2 500*c*	1.72
25*E*	3 800*t*	3 100*t*	1.23
17*E*	3 900*t*	3 000*t*	1.30
25*W*	3 900*c*	2 800*c*	1.39
19*W*	4 500*c*	2 900*c*	1.55
			Average, 1.44
"Over" Bars in Tension.			
16*W*	7 000	3 000	2.33
18*W*	7 000	2 200	3.19
20*W*	4 500	2 400	1.87
22*W*	5 700	2 200	2.59
24*W*	7 000	2 700	2.59
26*W*	3 800	2 200	1.73
			Average, 2.38

Fig. 16 gives the results of tests of the small columns. The dotted line represents the maximum fiber stress computed by considering the eccentricity of loading at any cross-section to be equal to the deviation of that section from a straight line connecting the ends of the column. The deflections were slight, and were neglected in the calculations. The solid line represents the stresses on the two sides of the column, as determined from the extensometer readings.

TABLE 14.—TENSION TESTS OF MATERIAL FROM COLUMNS.

Test piece from:	Material.	Number of test pieces.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Average Elongation. Percentage
Column No. 1 Angles................	Steel.	2	43 300	61 600	37
$\frac{7}{16}$ by $1\frac{1}{4}$-in. lacing bar............	Steel.	2	40 700	58 100	42
1 by $\frac{1}{4}$-in. lacing bar............	Steel.	1	42 400	62 000	38
Channels of wrought-iron columns.	Wrought iron.	3	30 700	46 800	17

Tests of Column Material.—Table 14 gives the results of the tensile tests of samples of material from various parts of the flange members of the columns, and also of tension tests of lattice bars. Table 15 gives tension tests of lattice bars like those used in the compression tests of lattice bars.

TABLE 15.—TENSION TESTS OF LATTICE BARS.

Shape.	Dimensions of cross-section of whole bar, in inches.	Number of specimens tested.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Percentage of elongation in 2 in.
Channel.	$1\frac{1}{2}$ by $\frac{3}{4}$ by $\frac{1}{8}$	1	43 000	57 600	36.5
Angle....	$1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{8}$	2	45 000	59 300	24.5
Flat......	$\frac{7}{8}$ by $\frac{7}{16}$	2	38 700	57 000	45.5
Flat......	1 by $\frac{3}{8}$	2	42 400	61 700	45.8
Flat......	$1\frac{1}{2}$ by $\frac{1}{4}$	2	44 600	60 800	42.8

COMPARISON AND DISCUSSION.

The Action of Built-up Compression Pieces.—In analytical discussions of column action, the stress is usually assumed to vary uniformly from a minimum on one side of the cross-section to a maximum on the opposite side, and the whole cross-section of the column is considered to act as a unit. The longitudinal axis of the column is also considered to take a definite elastic curve under load. In the

derivation of most column formulas, it is assumed that the amount of deflection of the elastic curve from the original position of the axis is an important element in fixing the maximum stress in the column. Although these assumptions are generally used as the basis of column formulas, it may be well to consider whether conditions may not exist, in columns of ordinary form and dimensions, which will render doubtful the general applicability of some of these assumptions, and will dwarf the effect of others. At any rate, it seems worth while to consider the effect of other conditions in a built-up member. It must be borne in mind that the built-up column is subject to imperfections of fabrication, and that some crookedness and eccentricity must exist.

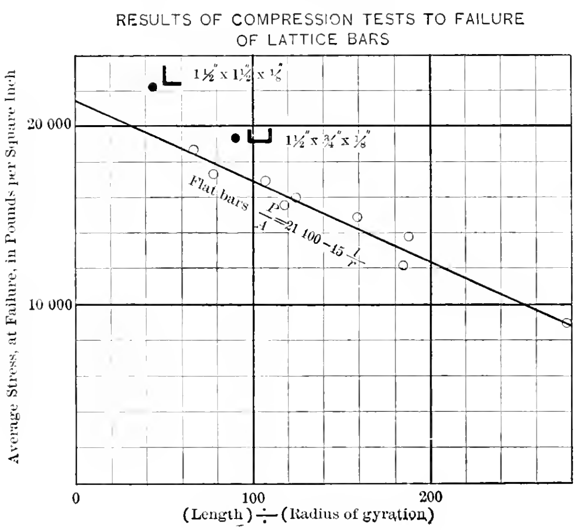


Fig. 18.

The component parts of the column may be relatively slender and flimsy. Whether there is integrity of cross-section under load, is a question. In the tests herein described, the amount of deflection from the original axis, for loads up to a point somewhat below incipient failure, was found to be slight (generally between 0.04 and 0.1 in.), much smaller than necessary to account for the stresses observed in the columns.

The action of short columns at failure may be expected to be different from that of longer columns, although the stresses up to incipient failure may be the same. Granting that the conditions of

non-straightness are such that the distribution of stress over the cross-section is the same for the two lengths of column, and that the deflection of the column is so slight as not to affect materially the stresses developed, the longer column will be in more danger of immediate and sudden collapse after the yield point of the material in any fiber has been reached, and the total load carried before complete failure will, in general, be less. This is because, in a ductile material, after the stress at one side of the column has passed the yield point, the total resistance of the section to compression will increase, while the resistance to cross-bending may not. Under the conditions named, the bending moment due to eccentricity will be the same until the yield point in some fiber is reached. After yielding begins, the greater deflection in the longer column rapidly increases its relative eccentricity, and more rapid failure may be expected than with the shorter column.

Indications of Data.—It will aid in the interpretation of the data of the distribution of stress over the channel members of the columns to point out a few simple indications. Reference may be made to the diagrams in Figs. 6, 7, 8, and 13, and to Tables 2, 3, 8, and 9.

1.—Any lack of agreement between the average load stress and the average of the stresses given for the four centers of gravity of channel flanges may be ascribed to errors of observation.

2.—If the stress at the center of gravity of one channel is above the average stress throughout the length of the column, and the corresponding stress for the other channel is similarly below the average stress, there must be an eccentricity in the application of the load at the two ends. If the stresses at the center of gravity of one channel member forms in the diagram a straight line which crosses the line of average stress, and that for the other channel crosses in the opposite way, the eccentricity of the load application must be oblique.

3.—If the stress at the center of gravity of a channel in near-by points is greater first in one channel and then in the other, the change may be due to crookedness of the column throughout that part of the length.

4.—If, in one channel or in one channel flange, the stress at the center of gravity remains constant and that of the extreme fiber varies, the change may be due to local crookedness of this channel and there will be a lateral bending of this member.

5.—If the front side of a channel has a higher stress than the back side, there must be bending action through its web, and *vice versa.*

6.—Changing stresses in the diagonally opposite corners of a channel may indicate twisting of the channel, and another combination of stresses may indicate a twisting or oblique distortion of the column as a whole.

An inspection of the diagrams shows that all these indications are found in the tests.

Does the Built-up Column Act as a Unit?—Engineers have often expressed doubt as to whether the parts of a built-up column act as a unit, although column formulas assume this unity of action. The tests throw some light on the question of the integrity of cross-section under load. The individual channel, of course, acts as a unit to resist bending action, though there are indications of twisting. The integrity of the whole section with reference to a plane parallel to the lacing seems probable, except as twisting action exists. With reference to a plane through the axis perpendicular to the plane of the lacing, this unity of action is not so certain. The tests on the distribution of compressive stress and likewise the cross-bending tests indicate that these built-up columns did not in all cases act as a unit but rather as two members not fully restrained by the lacing. The stresses in two channels at points in the same cross-section do not give the regularity of variation which would exist if the column bent as a unit. The elastic curve assumed by Column No. 1 under cross-bending load, shown in Fig. 2, Plate XI, differs from the computed elastic curve, though that for the wrought-iron column gives little difference. In the case of the posts of the White Heath Bridge, however, there is much closer agreement and a seemingly closer approach to unity of action.

Effect of Non-straightness of Built-up Columns Upon Distribution of Compressive Stress.—The effect of crookedness or other irregularities of a constituent member of a built-up column may be realized if a rough analysis of the case be made. Consider a part of one of the channels forming a column, taking the length between the connections of two adjacent lattice bars. This member is under compression. Owing to non-straightness or to the non-homogeneity of the material, the load on this short piece is not evenly distributed over the section;

that is, it is not centrally loaded, but may be considered to have an eccentricity with respect to the gravity axis. Call this eccentricity e (Fig. 19). Neglect any deflection of the piece under consideration due to the load. Call the compression load coming on this piece P; A its area of cross-section; I its moment of inertia about YY, and r the corresponding radius of gyration; and c the distance from YY

ECCENTRICITY IN CHANNEL MEMBER

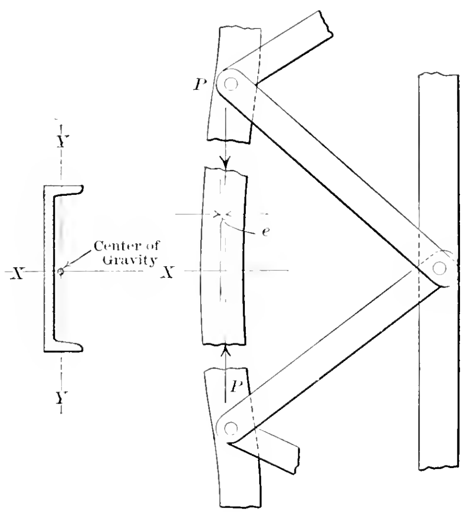


FIG. 19.

to the remotest fiber. Then the bending moment due to the eccentricity is Pe. The maximum stress will be

$$f = \frac{P}{A} + Pe\frac{c}{I} = \frac{P}{A}\left(1 + \frac{e\,c}{r^2}\right).$$

The excess of the stress in the extreme fiber of the piece over the average stress, produced by the eccentricity, e, is then $\frac{P}{A}\;\frac{ec}{r^2}$, and hence the term, $\frac{ec}{r^2}$, gives the proportionate excess of stress in the extreme fiber. This value is applicable to the channel, or to one flange of the channel, or it may be applied to the column as a whole by using the properties of the whole section. In the single channel under consideration, c is relatively large and r is relatively small, and the excess of maximum stress for a given eccentricity, e, may be expected

to be relatively large. It will be seen that for an excess of 50% in the extreme fiber of a channel of Column No. 1, e, by this formula, would be 0.045 in., and, in the wrought-iron columns, 0.057 in. A slight variation from straightness in a channel will account for considerable increase of stress.

Excess of Maximum Fiber Stress over Average Stress in Channel Members.—The diagrams and data show that the compressive stress is unevenly distributed over the cross-section of the columns tested, and also that there is great variation in this distribution at the various sections along the length of the column. It will be noted that in a number of sections the excess of stress was from 40 to 50 per cent. In one test of Column No. 1, an excess of 67% was found, and in the White Heath Bridge an excess of 73 per cent. Possibly these values were unusual, or the observations were erratic, but the indications of a fiber stress of from 40 to 50% in excess of the average stress were not uncommon.

It may be seen that among the causes to which the high fiber stress may be attributed are (*a*) non-straightness of the column as a whole, (*b*) non-straightness of the component channels, or eccentricity in the delivery of stress to them by the lacing, and (*c*) unknown eccentricity in the application of the load. It would be of interest to know how much of the increase of stress may be due to any one of these conditions. A study of the tests of Column No. 1 shows that generally only a small amount may be said to be due to non-straightness of the column as a whole. In but few cases is it found to be more than, say, 5 per cent. In four places it seems that the excess attributable to this may be estimated to be between 20 and 25 per cent. The effect of non-straightness of the individual channels seems to be greater. At several points the excess of stress attributable to this cause appears to be from 30 to 50 per cent. As already noted, a kink in the channels of 0.045 in. would give, by the analysis made, an eccentricity sufficient for a 50% increase in stress. Not all of this crookedness need be between adjacent rivet points, as the stress may not reach normal for some distance on either side. The effect of the third condition, eccentricity of application of the load, will vary with the construction. In Column No. 1 the effect of undetermined eccentricity of application of load appears to be not nearly as great as the effect of non-straightness of the component channels.

In the wrought-iron columns, which are much stockier, the lack of straightness in individual channels has less effect, seemingly less than 15%, and much the larger part of the high fiber stresses appears to be due to general column eccentricity or to eccentricity of loading.

The results for the posts of the White Heath Bridge are of interest in this respect. It is evident that the effect of non-straightness of channels was not very large, and also that effect of non-straightness in the column as a whole was relatively small. There is, however, an evident bending in the direction of the web of the channels. For example, in U_3L_3 South, the back side of the channels has the maximum stress at the top and the front side at the bottom. The bending moment producing this may be due to obliquity of end pressures or to a bending by the connecting floor-beam and top chord. A twisting action is also apparent. Post U_3L_3 North gave quite similar results.

Compressive Strength of Lattice Bars.—In the discussion of stress developed in column lacing, the stress considered was the average over the bar. As usually attached, there is considerable flexure in the bar, and the ability of the bar to carry this eccentric load should be considered. The bars are most likely to fail in compression, since they act as long columns eccentrically loaded.

The tests of individual lattice bars (Fig. 17 and Table 12) show that the maximum fiber stress may be several times the average stress. It is also seen that even in a short lacing bar the maximum load carried is only about one-half the yield point of the material. The necessity of using very low working loads on lattice bars appears to be important. It will be noted that at low stresses there is similarity of distribution of stress in the slender bars and in the thicker bars, but the slender bars fail at smaller computed fiber stress.

The results of tests to destruction of individual lattice bars are fairly well represented by the formula:

$$\frac{P}{A} = 21\,400 - 45\frac{l}{r}$$

where P = load at failure, in pounds, A = area of cross-section, in square inches, l is the distance, in inches, from center to center of rivet holes, and r is the radius of gyration, in inches, of the cross-section of the lattice bar. These results may be considered to be applicable to "under" lattice bars. For "over" bars it seems probable that the average stress at failure would be considerably less.

Effect of Cover-Plates and End Connections.—In the tests of the White Heath Bridge, the effect of the cover-plate seems striking. The upper chord, U_3U_4, composed of two built-up channels with one cover-plate, gave an excess fiber stress of 20% at the worst section, while the posts, composed of two channels laced on both sides, gave a maximum of 73 per cent. The high value in the posts may be due to other causes, but it seems reasonable to expect that the cover-plate will act to reduce the irregularities in fabrication.

The connections of the ends of the posts evidently exerted a very noticeable effect on the stress distribution. In one of the posts tested, the stress was greatest at one corner of the post at the top and at the diagonally opposite corner at the bottom. It will be remembered that the posts were riveted to the top chord, and were connected with the lower chords by pins. The floor-beams were riveted to the sides of the posts, and this connection affects the stress distribution. Readings of deformations taken on the floor-beams and posts show that the loaded beam was partly restrained at the ends by the post and that there was an appreciable bending in the post.

Stresses in Column Lacing.—If the load carried by one channel of a column were the same throughout its length, no stress would be carried by the lattice bars. Such stress is developed whenever there is a change in the relative amount of loads carried by the two channels. If at the section, *AB* (Fig. 2), there is an equal division of load between the two channels, and also at the section, *CD*, and if at some section, *EF*, the division of load is unequal, it is evident that the lattice bars must be called into action to transmit this stress, and that transverse shear exists in the interval. In general, the conditions producing this must be complex, rendering analysis unsatisfactory, except in so far as the shear may be due to a known eccentricity of loading.

It is evident from the tests that the relative stress in the two channel members varies considerably from end to end and that the stress in lattice bars also varies. It seems probable that the transverse shear developed may be traced largely to irregularities in outline, or at least that these irregularities may be expected to cover up other causes of stress in the lacing of centrally-loaded columns, if we include in such irregularities all unknown eccentricity.

The amount of transverse shear necessary to produce the maximum

observed lattice-bar stress (given in Table 6) is of interest, though of course it cannot be taken to be conclusive. The measurements were generally made at working loads. So far as observations were made on columns tested to failure, the distribution of stress remained much the same up to incipient failure. The values given in Table 6 indicate maximum average stresses in the bars such as would be caused by a transverse load ranging from 2 to 6% of the central compression load or of a transverse shear one-half as great.

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PAPERS AND DISCUSSIONS

CAISSON DISEASE AND ITS PREVENTION.

By Henry Japp, M. Am. Soc. C. E.
To be Presented June 2d, 1909.

The prevention of caisson disease has not received as much attention as its cure, the general opinion being that it cannot be prevented. In forming an idea as to the possibility of its prevention, something may be learned from Nature's method of carrying out work by the aid of living organisms under air pressure.

First let us inquire whether Nature has a sufficient range of air pressure to make it necessary for a capacity to sustain variations without injury. The highest altitude on this earth is Mount Everest, 29 000 ft. above the level of the sea, and the deepest mine is Tamarack No. 3 Shaft, of the Calumet and Hecla Mines, 4 407 ft. below sea level.

The curve on Fig. 1 shows the absolute air pressures at various altitudes between these extremes. The total difference of 12½ lb. per sq. in., or from 9¾ lb. below atmospheric pressure at sea level (viz., 14.7 lb.) up to 2¾ lb. above normal, is considerable.

Mount Everest has never been ascended, but an elevation of 20 000 ft. has been reached, and men can live in comfort at an altitude of 7 000 ft., as in Mexico City, which has an air pressure of 3½ lb.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

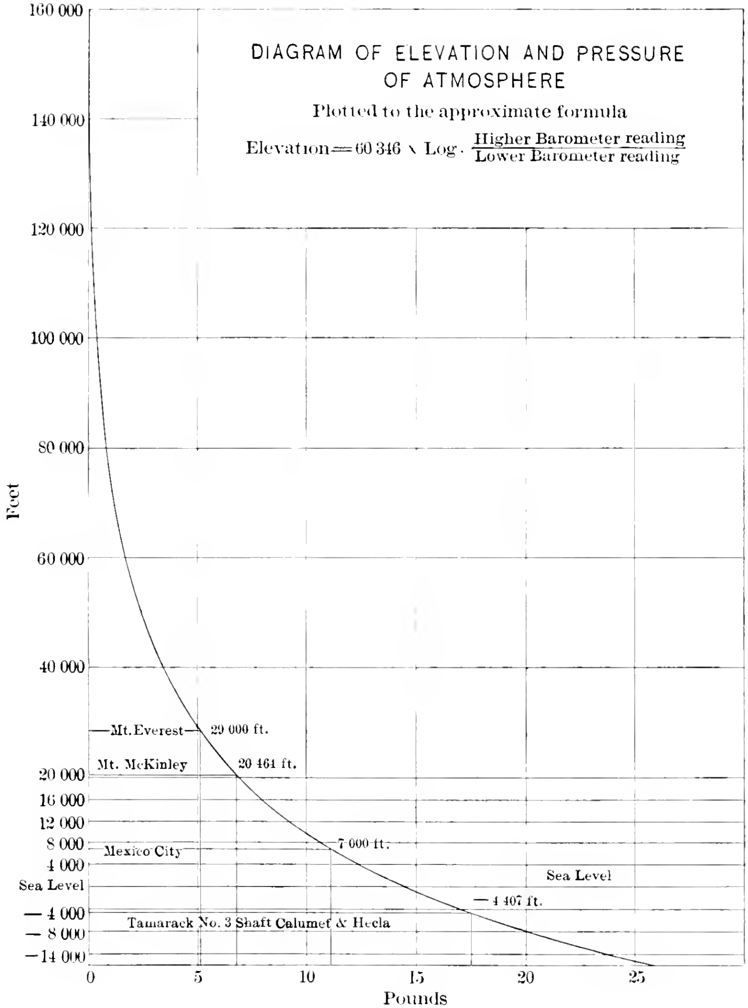


DIAGRAM OF ELEVATION AND PRESSURE OF ATMOSPHERE
Plotted to the approximate formula
Elevation = 60 346 × Log. Higher Barometer reading / Lower Barometer reading
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
16 000
12 000
8 000
4 000
Sea Level
— 4 000
— 8 000
—14 000
Feet
Mt. Everest
29 000 ft.
Mt. McKinley
20 464 ft.
Mexico City
7 000 ft.
Sea Level
— 4 407 ft.
Tamarack No. 3 Shaft Calumet & Hecla
0
5
10
15
20
25
Pounds

FIG. 1.

per sq. in. less than at the coast. Such a change of pressure is quite noticeable to those whose Eustachian tubes are closed, and, although the range of pressure is insufficient to produce caisson disease, yet travelers have complained of discomfort, not unlike a very mild attack, after a rapid journey by rail from Vera Cruz to Mexico City.

Eagles are known to go to great heights beyond the range of human vision, fish are capable of going to great depths in water, and beasts of prey roam from the mountain side to the valleys far below. With each descent these creatures have undergone compression, and with each ascent decompression.

It will be noted that Nature has no time limit for immersion under increased pressure, and the application of pressure is a much speedier and easier operation than that of decompression; and, while the compression is down-hill and attained with little effort, the decompression can only be attained by the continuous effort of climbing, and, in long ascents, with frequent rests or by stages. This is surely a forecast of the stage-decompression theory.

From Nature then, our best teacher, we learn that the time taken to enter the air-chamber may be short, the time spent in the air-chamber may be long, and the rate of coming out must be slow and accompanied by exercise.

In early compressed-air work engineers were confronted with caisson disease, and have long been striving for a remedy. Men of observation noted first that lock tenders who made frequent entry to the air-lock for short visits were much more immune from the malady than workers who stayed for a full shift, and from this argued that shortening the time in the air-chamber would be a safeguard, and, as higher pressures were used, the hours continued to be shortened until it was thought that half an hour at a maximum of 50 lb. gauge pressure was the limit of human endurance.

As the number of compressed-air works increased, so the workmen gained in experience, and it was noted that workmen of experience who suffered from the painful forms of caisson disease relieved the pain by re-entering the air-chamber.

About thirty years ago Paul Bert advanced the theory that caisson disease was caused by the nitrogen of the air being dissolved in the blood. He advocated slow decompression, suggesting 30 min. for 2 to 3 atmospheres (15 to 30 lb. gauge pressure) and 60 min. for 3 to 4

MEDICAL AIR LOCK USED IN THE EAST RIVER TUNNELS OF THE PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD

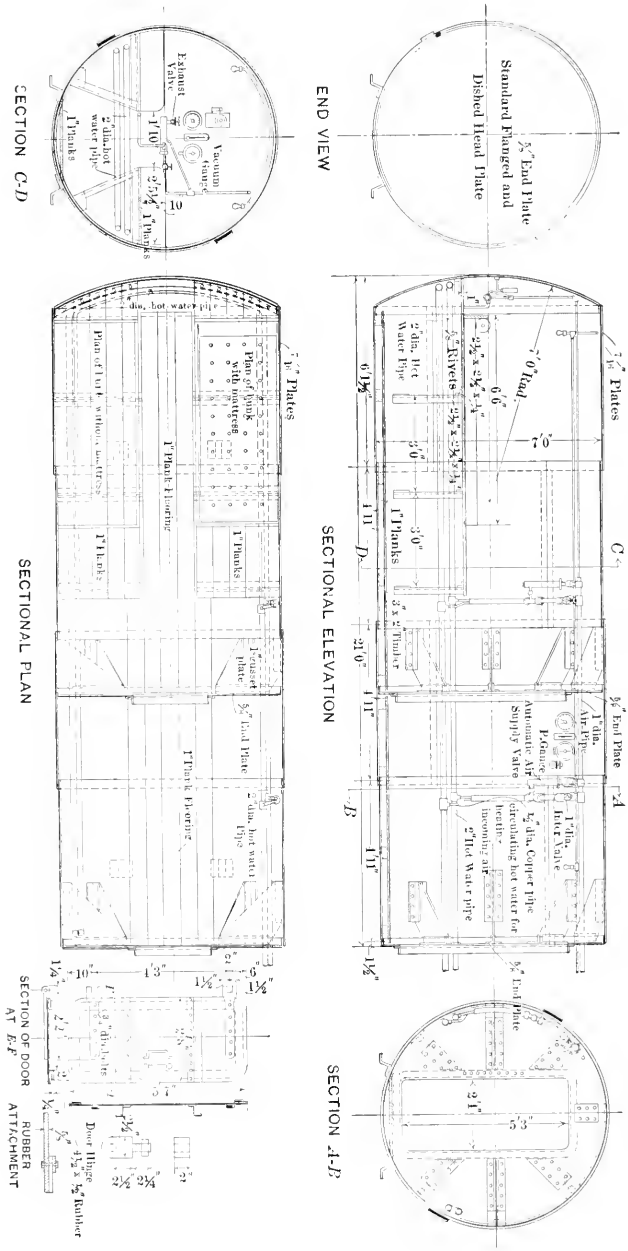


FIG. 2.

atmospheres (30 to 45 lb. gauge pressure). Since his time many writers and experimenters have followed in his footsteps, but it was not until Dr. Smith, of New York, suggested that a recompression chamber would act as a remedy, and E. W. Moir, M. Am. Soc. C. E., of London, without knowledge of Dr. Smith's prior suggestion, actually built a medical air-lock, or recompression chamber, in 1890, on the old Hudson Tunnel, New York, that any reliable cure for the trouble was discovered. Fig. 2 shows the medical air-lock designed by Mr. Moir, of which six were used on the East River tunnels of the Pennsylvania Tunnel and Terminal Railroad.

It was then found that if a man suffering from caisson disease in any of its many forms, varying from unconsciousness and paralysis to an acute pain in the limbs, was quickly recompressed and allowed to decompress slowly, in many cases a cure resulted, but not always.

This went to prove that the disease was a mechanical one, and was caused by the dissolved air in the blood and tissues becoming liberated in the body in the form of expanding bubbles which tore the tissues, caused pressure on the brain, injured the spinal cord, or frothed up the blood and stopped the circulation and heart action.

Much has been done since the introduction of the medical air-lock. The percentage of carbon dioxide in the air of the working chamber has been kept within safe limits, moisture and oil have been extracted from the air before delivery to the chamber, and the workmen have been well cared for; but still caisson disease claims its victims, and engineers consider this inevitable.

The erratic manner of the disease puzzles doctors and engineers alike. Men who have worked for months in high pressures for the regulation shift without suffering have been suddenly attacked and died; others, working only half a shift, have been paralyzed; while exceptions have worked for 12 hours at a time and have suffered only slight pains which have quickly passed away.

The writer, as Managing Engineer for S. Pearson and Son, Inc., on the East River Tunnels contract for the Pennsylvania Tunnel and Terminal Railroad Company, New York, with as many as ten tunnel headings under compressed air simultaneously, has had many opportunities of studying this strange disease.

When these tunnels were commenced, in 1904, the knowledge of compressed air at the disposal of the contractor was quite extensive,

and included the long experience of Alfred Noble, Past-President, Am. Soc. C. E., on caisson work in the United States, and Mr. E. W. Moir's experience on the Forth Bridge caissons, the old Hudson Tunnel, and the Blackwall caissons and tunnel.

In the light of past experience, the general conduct of the work was framed on a few established rules which when condensed amount to the following: No workman was allowed to enter the air-chamber without a physical examination by the qualified medical officer of the contractors. Sound physique was the chief requirement. The men were cautioned not to enter the air on an empty stomach, to wear warm clothing on coming out, and to drink hot coffee.

The time worked in the air-chamber was limited to 8 hours with half an hour off for lunch, up to 32 lb. gauge pressure, and two spells of 3 hours each with 3 hours rest between for pressures from 32 to 42 lb., and two spells of 2 hours each for pressures greater than 42 lb. with 4 hours rest between, with no limitation as to decompression. Two medical air-locks were installed on each side of the river, well-warmed dressing rooms were provided for the workmen, and there were covered gangways for access to the shafts.

The air was cooled before delivery to the tunnels, and samples were taken in the tunnels by Mr. Noble's engineers and analyzed daily. The air was regulated so that the carbon dioxide did not exceed 10 parts in 10 000, and the tunnels were kept in a sanitary condition.

Owing to the grade of the tunnels being so deep on the Manhattan side of the river, the air pressure very quickly rose to 36 lb.

Practically no cases of bends occurred until the pressure reached 29 lb., and then, within a few days of each other, two men died. These men had entered the air-chamber without being passed by the doctor. Then it became necessary to post outside each air-chamber a guard whose duty it was to keep out of the tunnel men who had no doctor's pass. At this time, reliance could not be placed on the tunnel foremen, as they were likely to be absent from work, and new men had to be selected each day.

For many months after the work started, while the men were being seasoned, the tunnel gangs and foremen were in a state of change, owing to the difficulty of getting good men and the frequent absences due to caisson disease, and it was a long time before an efficient organization was built up.

As the tunnels were driven deeper beneath the East River the pressure quickly rose, and ultimately reached 36 lb., gauge pressure, with only one set of air-locks in operation; but even the change at 32 lb. from one shift of 8 hours to two shifts of 3 hours each gave no relief, and cases of bends, sometimes fatal, continued all the time.

It was not long after 27 lb. was reached that the more sensitive members of the staff found that it did not pay to come out quickly, and at 30 lb. pressure it became a custom to take about ½ min. for each pound. After one or two additional fatal cases occurred, it was decided to limit the workmen to approximately the same rate of decompression, or actually 15 min. for 35 lb. pressure.

Many of the men complained that taking so long to decompress gave them caisson disease, and it was difficult to compel them to take long enough. The guards at the entrance to the tunnels had now to record the time taken to decompress, and, as the workmen frequently used the lower muck locks as well as the upper man lock, it was impossible to tell when decompression commenced owing to the noise of exhausting air. Therefore it became necessary to run a small ¼-in. pipe from the exhaust pipe of each lock to the cabin in which the guard was stationed, a whistle was attached, and a small ball of cotton was suspended by a light string over each pipe. The clerk, noting when the ball was puffed off for each lock, booked the workmen off as they left the lock. The material locks were fitted with inner material valves and, in addition, man valves of smaller size, and a pressure gauge and a clock were fixed in each air-lock.

As the guard had already booked the men as they entered, he could tell if any were exceeding the regular working shift, and his record was valuable for checking the time-keepers. This rough method of checking the duration of the decompression was quite good enough for the purpose. An attempt to improve it was made when a 12-in. Crosby recording gauge was installed on one air-lock, but it was ineffectual because the air-lock was often sent out or decompressed with no one inside, and this complicated the record, which was much too small, and involved considerable trouble in locating the record of decompression for individual gangs. No doubt a suitable recording instrument could be devised for this special purpose.

The effect of lengthening the decompression period to 15 min. reduced the number of cases of bends, and no doubt prevented many

fatal ones, but they still occurred. As the tunnels had many months to run at high air pressures, the question was: What else can be done to prevent them?

The writer, on coming out of the tunnel with the workmen, observed that the rate of decompressing was most irregular. One lock tender would allow the air to escape slowly until the 15 min. was almost exhausted, and then, by opening the valve, would let off the remaining pressure very quickly. Others would reverse the process, and exhaust quickly, and then keep the men under 2 or 3 lb. until the time expired. To avoid this, a simple decompressing valve was designed (Fig. 3) which gave a uniform decompression from 35 lb. to atmosphere in 15 min. A somewhat similar one (Fig. 4) was designed for the medical air-lock, for 1 hour decompression for 35 lb., with an automatic ventilator attached.

These valves certainly improved conditions, but still fatal cases occurred. After the first valve was under operation, the writer's attention was called to "Modern Tunnel Practice," by D. McN. Stauffer, M. Am. Soc. C. E., wherein a description is given of a needle decompression valve used on the air-locks at the Kiel Dry Dock Works, in Germany. A similar valve was also used for compression. On entering the lock the air was admitted at the rate of 1.5 lb. per min., and was decompressed at the rate of $\frac{3}{4}$ lb. per min., or, for 35 lb. gauge pressure, 23 min. for entering the air and 46 min. for leaving. This needle-valve was often frozen up, but otherwise worked well.

Such speeds seemed altogether too slow, and Mr. Stauffer in his book states that "these rates would be deemed excessively slow in American caisson practice." No date is given for the Kiel Dry Dock work, but presumably it was under way in 1904.

It was not thought advisable to increase the time of decompression at that time, but preliminary tests under air pressure in the tunnels for $1\frac{1}{2}$ hours were then tried for green men, followed by a second medical examination after decompressing in 15 min. A few men were eliminated by this test, and one case of permanent paralysis resulted from the test in 34 lb. gauge pressure. Fresh starters were made to stay in the tunnel for one-quarter of a shift for the first half, and if no caisson disease followed they were allowed to work for the second half. This proved a good safeguard.

AUTOMATIC CONSTANT RATE COMPRESSION VALVE

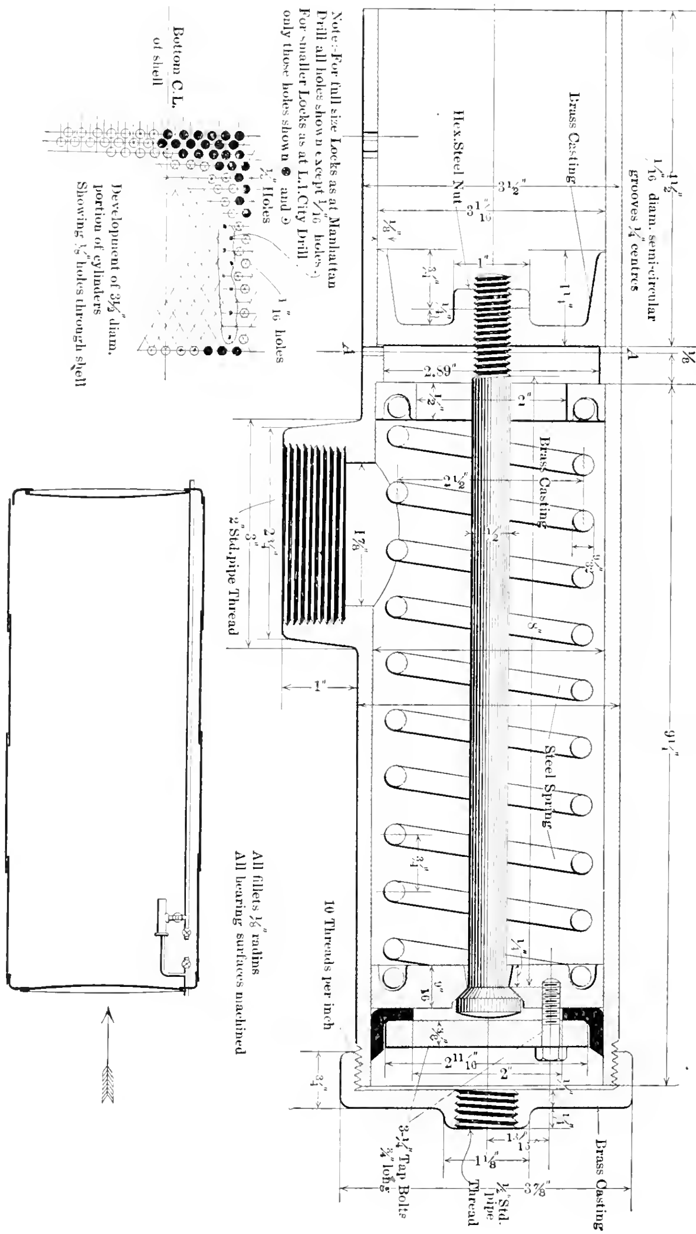


DIAGRAM OF CONNECTIONS

FIG. 3.

AUTOMATIC CONSTANT RATE DECOMPRESSION AND VENTILATING VALVE

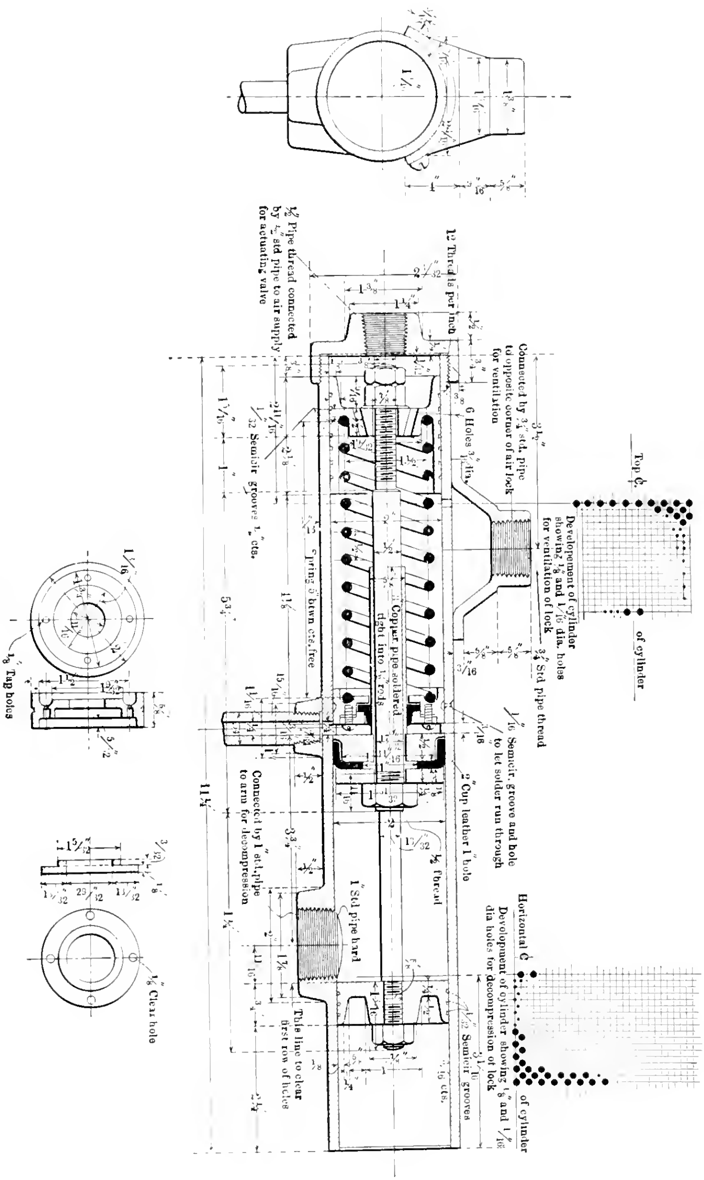


FIG. 4.

On November 8th, 1906, a second bulkhead was put in operation in one of the tunnels, and the pressure between the two bulkheads was reduced to 15 lb. gauge. The number of cases of disease was very small for that tunnel, and as soon as possible additional bulkheads were placed in all four tunnels. The result was to have been expected from the experience in other tunnels where two bulkheads were used; the exercise of the long walk between bulkheads, at low pressure, seemed to assist in driving off the bubbles of air from the blood.

The workmen were allowed to decompress from 35 to 15 lb. as quickly as they pleased. They then walked for 500 ft. along the tunnel under 15 lb., taking at least 5 min., and then decompressed from 15 lb. to atmosphere in 10 min., so that in all about 16 min. were occupied in decompressing. When the inner pressure was less than 32 lb. an 8-hour shift was worked, with ½-hour interval for lunch, between bulkheads in low air pressure.

Just when it looked as if the double bulkheads with stage decompression had eliminated fatal and severe cases of caisson disease, two deaths occurred in physically perfect subjects.

In order to discover, if possible, the connection between the cases of disease and other things, charts were plotted by the medical staff for some months, showing the rise and fall of air pressure, hours worked, humidity, temperature, percentage of carbon dioxide, and number of green men in the tunnel, along with barometer readings, condition of weather, direction of wind, and number of cases of bends. The results were not very encouraging, but it was noted that the number of green men, the height of pressure in the tunnel and the number of cases of bends varied together.

The percentage of cases in air pressure of 31½ lb. for 8-hour shifts was no more than the percentage in 32½ lb. for two 3-hour shifts—in fact, it was, if anything, less for the longer shift. The decrease in length of shift added one extra gang of men, and probably many of these men being green accounted for this.

In November, 1906, Mr. Moir became acquainted in part with Dr. Haldane's work of investigation for the British Admiralty on "Deep Sea Diving," and ordered the decompressing valves changed so that the decompression was accelerated at the commencement of the operation, and slowed off toward the end. In October, 1906, the writer, endeavoring to find out the effect of sudden decompression,

fitted an ordinary glass siphon bottle with a valve and pressure gauge. This bottle was partly filled with water, and put under 36 lb. air pressure for 12 hours. On suddenly releasing the pressure, no effervescence whatever took place, and only a very few minute bubbles were visible. The same thing was tried with the bottle partly filled with bullock's blood, with like results. Arguing that the placid surface of blood in a stationary vessel offered no such opportunity for dissolving air as occurs in the lung surface with the blood circulating, the siphon was again charged with 36 lb. pressure, but was rotated for 24 hours under this pressure. The same result as before was observed on suddenly releasing the pressure; but, on closing the valve, the pressure recorded on the gauge was about 1 lb. per sq. in. in less than 1 hour, and in 6 days a pressure of 13 lb. was recorded, showing that the air dissolved in the blood came off very slowly.

If the cork of a bottle of aerated water is drawn quickly, it effervesces so violently that the water froths out of the neck of the bottle; but, if the gas is allowed to escape slowly, only a mild bubbling results. This in general is the theory on which slow decompression has been advocated in the past. The violence of the escape of carbon dioxide gas from aerated water, as compared with the escape of air from water released from pressure, is due to the greater solubility of carbon dioxide than air.

It was difficult to know what else could be done at this time in the way of eliminating caisson disease on the East River tunnels, and the cases of disease were certainly less frequent and less severe, although at long intervals fatal cases still occurred. The workmen, by this time, had become seasoned, and more seasoned men were now available from other works.

Dr. Leonard Hill read a paper on deep sea diving and caisson work, on September 26th, 1907, and exhibited the web of a living frog under a pressure of 20 atmospheres. On suddenly decompressing, bubbles were plainly visible in the blood vessels, multiplying quickly as they traveled along until the circulation was choked; on recompression the bubbles disappeared and circulation was resumed. He stated that he and Mr. Greenwood had been under an air pressure of 92 lb., and had suffered no ill effects.

Dr. J. S. Haldane, on November 29th, 1907, read a most enlightening paper before the Society of Arts, in London, in which he gave

the results of his investigation for the British Admiralty on "Deep Sea Diving."

He set forth very clearly the question of the solution of gases by the blood, fat, and tissues of the body, and attempted to determine the time taken to saturate the blood with the nitrogen of the air. He proved conclusively that the quantity of carbon dioxide adjusts itself to a constant percentage inversely proportional to the pressure in the alveoli, or air cells.

In atmospheric pressure, the percentage of carbon dioxide in the alveolar or expired air is 5.6%, and at a pressure of 2 atmospheres, absolute, this is reduced to 2.8%; so that the question of the percentage of carbon dioxide in the air of the working chamber is not important unless it approaches the percentage of carbon dioxide in the air cells of the lungs.

For instance, if the air-chamber is under an air pressure of 30 lb., or 3 atmospheres absolute, the percentage of carbon dioxide in the air cells is 5.6 divided by 3, or 1.86%; and, if the percentage of carbon dioxide in the air-chamber does not exceed 1%, no ill effects will arise. This is ten times as much as is generally specified, namely, 10 parts in 10 000, and greatly reduces the amount of compressed air necessary per man per hour, which can be calculated approximately from the following simple formula:

$$\text{Cubic feet per man per hour} = \frac{\text{80 cu. ft. permitted above normal air}}{\text{Percentage of rise of } CO_2}.$$

Thus, if 0.04% is the CO_2 in the atmosphere, and the percentage in the tunnel is allowed to go up to 0.10%, the air required per man per hour $= \dfrac{80}{0.06} = 1\ 333$ cu. ft.

Dr. Haldane has completely removed all difficulty of respiration and discomfort of divers at great depths through the application of this theory regarding the percentage of CO_2 by supplying the diver with the same volume of compressed air per minute at different levels instead of the same volume of free air per minute.

The question of ventilating compressed-air tunnels seldom arises, as the volume of air generally far exceeds the amount required for purity, on account of the tremendous leakage at the face, so that this is not nearly as important as is indicated by Dr. Haldane's investigation of the solubility of air by the blood and tissues of the body.

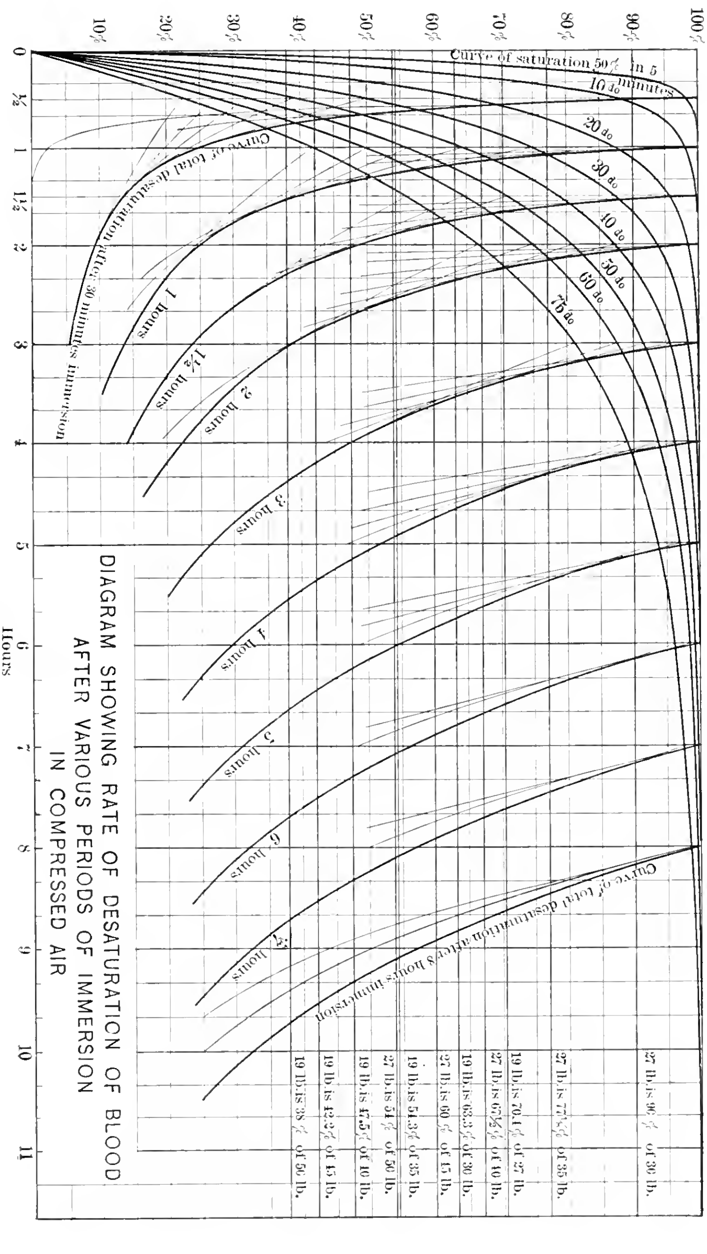


DIAGRAM SHOWING RATE OF DESATURATION OF BLOOD
AFTER VARIOUS PERIODS OF IMMERSION
IN COMPRESSED AIR
Hours
Curve of saturation 50% in 5 minutes
10 do
20 do
30 do
40 do
50 do
60 do
75 do
Curve of total desaturation after 30 minutes immersion
Curve of total desaturation after 8 hours immersion
1 hours
1½ hours
2 hours
3 hours
4 hours
5 hours
6 hours
7 hours
27 lb. is 90% of 30 lb.
19 lb. is 70.4% of 27 lb.
19 lb. is 63.3% of 30 lb.
27 lb. is 60% of 45 lb.
19 lb. is 54.3% of 35 lb.
27 lb. is 54% of 50 lb.
19 lb. is 47.5% of 40 lb.
19 lb. is 42.2% of 45 lb.
19 lb. is 38% of 50 lb.

Fig. 5.

After demonstrating that the CO_2 is dealt with automatically, he shows that nearly all the oxygen dissolved by the blood is taken up chemically by the hemoglobin, but the nitrogen remaining is incapable of combination with the system, and is dissolved by the blood and tissues. These, therefore, become saturated with nitrogen during immersion in compressed air.

If saturation takes place quickly it will not matter how long a workman stays under air pressure; but, if slowly, then shortening of the working shift will be a safeguard.

The rate of saturation is the same as the rate of desaturation if compression and decompression are instantaneous; so that, by altering the time for decompression, it is possible to determine by experiment the rate of saturation and desaturation.

Dr. Haldane experimented with men and goats at high air pressures, with varying lengths of stay under air pressure, and with different speeds of decompression. From these he concluded that certain parts of the body, where the circulation is rapid and the number of blood vessels high for the mass of the part supplied, would be half saturated or desaturated in 5 min., while other parts, with slight circulation for the mass, would require 75 min. for 50% saturation, especially the fatty parts, as fat is found to dissolve about six times as much nitrogen as blood.

Fig. 5 shows the rate of saturation, on the basis of half saturation in various times from 5 to 75 min., and curves of desaturation, if the decompression is instantaneous after varying times of immersion from ½ hour up to 8 hours.

It will be seen that 90% saturation takes place in the slowest parts of the body in about 4 hours, and complete saturation for the quickest parts in about 40 min., so that after 40 min. immersion there is danger in too rapid decompression.

Dr. Haldane has noted that no serious cases of caisson disease have occurred for rapid decompression at a working pressure of 19 lb. gauge pressure, or 2.3 atmospheres, absolute. Therefore, he concludes that it is always safe to decompress rapidly to half the absolute pressure, or even to the absolute pressure divided by 2.3, without bubbles being liberated. Further, if the decompression for the remaining pressure is continued at such a rate as will keep pace with the rate of desaturation, then, the absolute pressure of nitrogen in the blood never exceed-

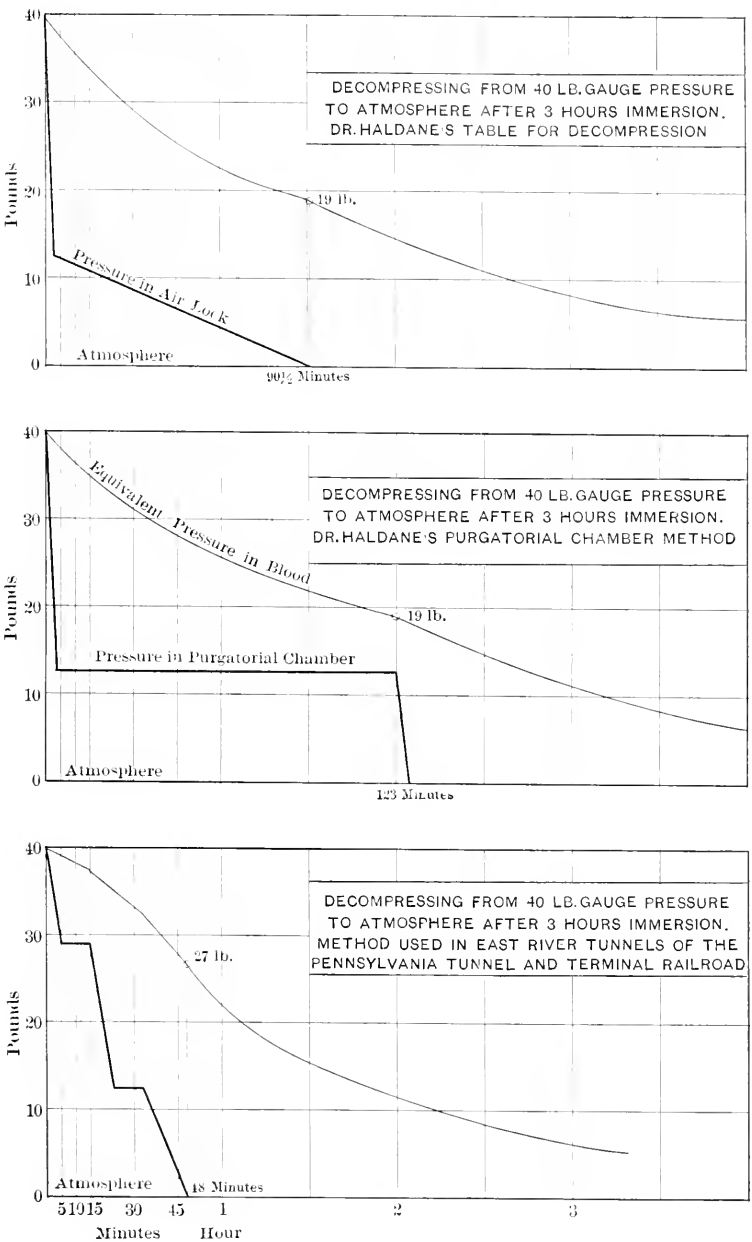

DECOMPRESSING FROM 40 LB. GAUGE PRESSURE
TO ATMOSPHERE AFTER 3 HOURS IMMERSION.
DR. HALDANE'S TABLE FOR DECOMPRESSION
Pounds
40
30
20
10
0
19 lb.
Pressure in Air Lock
Atmosphere
90½ Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE
TO ATMOSPHERE AFTER 3 HOURS IMMERSION.
DR. HALDANE'S PURGATORIAL CHAMBER METHOD
Pounds
40
30
20
10
0
Equivalent Pressure in Blood
19 lb.
Pressure in Purgatorial Chamber
Atmosphere
123 Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE
TO ATMOSPHERE AFTER 3 HOURS IMMERSION.
METHOD USED IN EAST RIVER TUNNELS OF THE
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
Pounds
40
30
20
10
0
27 lb.
Atmosphere
48 Minutes
5 10 15 30 45 1 2 3
Minutes Hour

Fig. 6.

ing 2.3 times the absolute pressure in the air-lock during decompression or after returning to the atmosphere, no symptoms will occur.

Table 1 shows the rates of decompression advocated by Dr. Haldane for caisson workers:

TABLE 1.—DR. HALDANE'S RATE OF DECOMPRESSION IN CAISSON AND TUNNEL WORKS.

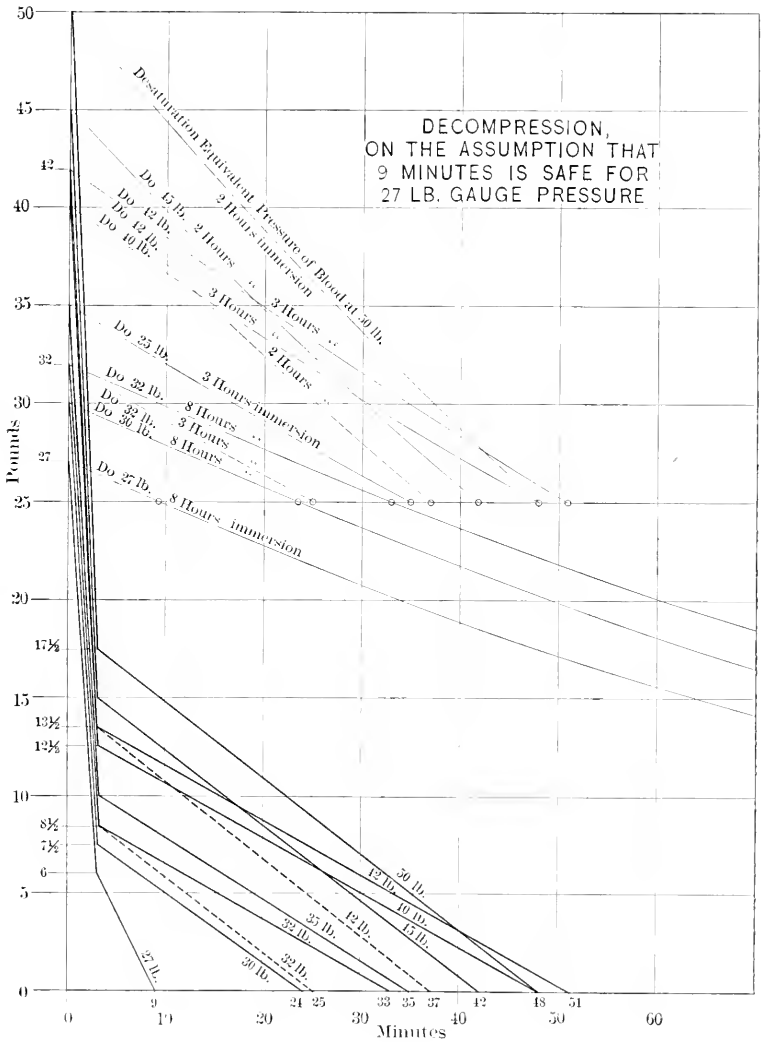
Working pressure, in pounds per square inch.	NUMBER OF MINUTES FOR EACH PERIOD OF DECOMPRESSION AFTER THE FIRST RAPID STAGE.		
	After first 3 hours' exposure.	After second or third 3 hours' exposure showing an interval for a meal.	After 6 hours or more of continuous exposure.
18–20	2	3	5
21–24	3	5	7
25–29	5	7	8
30–34	6	7	9
35–39	7	8	9
40–45	7	8	9

If the pressure is 40 lb., gauge, decompress rapidly from 40 lb. to $\frac{40 + 15}{2} = 27\frac{1}{2}$ lb., absolute, or $12\frac{1}{2}$ lb., gauge, in 3 min., and then take 7 min. for each remaining pound, viz., $12\frac{1}{2}$ lb., or $87\frac{1}{2}$ min. plus 3 min. $= 90\frac{1}{2}$ min., or $1\frac{1}{2}$ hours in all for a 3-hour immersion; or, for a 6-hour immersion, 115 min. in all.

The upper diagram on Fig. 6 shows this graphically.

It will be seen that, if this rule is followed, the condition of the absolute pressure, being half the pressure in the blood, is obtained. If uniform decompression for the same time were adopted, the decompression would be too slow for the earlier part, would very much retard the desaturation, and would give a greater tension in the blood than 19 lb., on coming out of the air pressure.

It is obvious that while the air-lock has any pressure in it the desaturation takes place only in relation to that pressure, and, if one adopts Dr. Haldane's idea of keeping a portion of the tunnel at half the absolute pressure for a dressing room to act as a purgatorial chamber, where the men may wash and change while desaturating down to 19 lb., we must make a new curve for desaturation and increase the time of decompressing, as will be seen by the second diagram on Fig. 6.



DECOMPRESSION, ON THE ASSUMPTION THAT 9 MINUTES IS SAFE FOR 27 LB. GAUGE PRESSURE
Desaturation Equivalent Pressure of Blood at 50 lb. 2 Hours immersion
Do 45 lb. 2 Hours
Do 42 lb. 3 Hours
Do 42 lb. 2 Hours
Do 40 lb. 3 Hours
Do 35 lb. 3 Hours immersion
Do 32 lb. 8 Hours
Do 32 lb. 3 Hours
Do 30 lb. 8 Hours
Do 27 lb. 8 Hours immersion
50 lb.
42 lb.
40 lb.
45 lb.
42 lb.
35 lb.
32 lb.
32 lb.
30 lb.
27 lb.
Pounds
Minutes

FIG. 7.

From this it appears that the tension in the blood does not reach 19 lb. until a period of 123 min. has been passed in the purgatorial chamber after 3 hours' immersion.

It will be noticed that Dr. Haldane bases his theory of decompression on the fact that no cases of caisson disease are chronicled for men working in gauge pressures up to 19 lb., or 34 lb., absolute, and if it is safe to decompress suddenly from 34 lb., absolute, to 15 lb., absolute, it is safe to decompress suddenly from 50 lb., absolute, to 22 lb., absolute, or from 35 lb., gauge pressure, to 7 lb., gauge pressure.

The question arises at once: Are the times suggested by Dr. Haldane practical?

Many caissons of small dimensions are sunk under air pressures of 35 lb. and to cramp men in a small air-lock for 73 min. is out of the question, and the fitting of large compartments in small vertical air-locks, as used in building foundations in New York City, would be difficult. In tunnel work it would be easier, but the present method of rapid decompression is so radically different from Dr. Haldane's suggestion that it quite appals one to think of taking so long.

The experience of most compressed-air works is that up to 27 lb. gauge pressure there is very little trouble, and using this as the safe limit the times required to reduce the saturation in the blood to 27 lb. is not so excessive for pressures up to 50 lb.

The horizontal lines on Fig. 5 show the equivalent points for 27 and 19 lb. in the blood for 50, 45, 40, 35, 30, and 27 lb., gauge pressure.

It will be noticed that the slowly saturated parts of the body are only partly saturated in the first few hours of immersion, and therefore the desaturation is proportionately fast.

From Fig. 5, Tables 2 and 3 have been compiled; they show the risks taken in rapid decompression after long immersion.

If the decompression is slow, these curves will be flattened out, and a longer time will elapse before the saturation of the blood falls to the equivalent of 19 lb. or 27 lb., as the case may be.

Fig. 7 shows the curves of decompression from 27, 30, and 32 lb., gauge, after 8 hours' immersion, and for 32, 35, 40, and 42 lb., gauge, after 3 hours' immersion, and 42, 45, and 50 lb., gauge, after 2 hours' immersion, on the basis that it is safe to decompress from 27 lb., gauge pressure, in 9 min. This reduces the saturation in the blood to the equivalent of 25 lb. on reaching atmospheric pressure.

TABLE 2.—TIMES (IN MINUTES) REQUIRED FOR PRESSURE IN BLOOD TO FALL TO THE EQUIVALENT OF 19 POUNDS GAUGE PRESSURE WITH INSTANT DECOMPRESSION AFTER VARIOUS PERIODS OF IMMERSION IN COMPRESSED AIR.

No. of hours of immersion.	GAUGE PRESSURES:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour.............	4	5	8	10	14	16½
1 "	7	10	15	20	26	32
1½ hours	10	16	24	32	40	48
2 "	14	21	32	42	52	62
3 "	22	30	45	59	72	82
4 "	28	39	56	69	82	93
5 "	33	44	60	74	86	98
6 "	35	47	63	76	89	100
7 "	37	48	64	78	90	102
"	38	50	66	79	92	103

The times in Table 2, being for instant decompression, must be suitably extended for slow decompression.

TABLE 3.—TIMES (IN MINUTES) REQUIRED FOR PRESSURE IN BLOOD TO FALL TO THE EQUIVALENT OF 27 POUNDS GAUGE PRESSURE WITH INSTANT DECOMPRESSION AFTER VARIOUS PERIODS OF IMMERSION IN COMPRESSED AIR.

No. of hours of immersion.	GAUGE PRESSURES:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour.............	......	1	3	4	6	8
1 "	......	1½	5	8	12	16
1½ hours	......	2	6	12	18	24
2 "	......	3	9	17	25	32
3 "	......	5	15	25	34	45
4 "	......	6	19	32	45	57
5 "	......	8	22	37	50	61
6 "	......	9	25	40	52	64
7 "	......	10	27	41	54	65
8 "	......	11	28	43	55	66

The times in Table 3, being for instant decompression, must be suitably extended for slow decompression.

Table 4 is taken from Fig. 7, and in part is confirmed by the experience in the East River tunnels as comparatively safe. Caisson disease will result, but no fatal or severe case should be experienced with physically sound men if these times are adhered to.

TABLE 4.—DECOMPRESSION TABLE BASED ON 9 MINUTES BEING SAFE FOR 27 POUNDS GAUGE PRESSURE.

Gauge pressure, in pounds.	Reduce pressure in 3 minutes to:	Total time in air-lock after 8 hours' work:	Total time in air-lock after 3 hours' work:	Total time in air-lock after 2 hours' work:
27	6 lb.	9	....	....
30	7½ "	24	....	....
32	8½ "	33	25	....
35	10 "	....	35	....
40	12½ "	....	48	....
42	13½ "	....	51	37
45	15 "	....	....	42
50	17½ "	....	....	48

Some time after reading Dr. Haldane's paper and studying his theory, it became necessary to raise the pressure in the tunnels to 40 lb., gauge. It was possible to make the workmen pass through three sets of air-locks on leaving the tunnel. The inner chamber was kept at 40 lb., the intermediate chamber at 29 lb., and the outer chamber at 12½ lb.

The men were ordered to take 5 min. in the first lock, 8 min. in the second lock, and 15 min. in the third. There was a distance of approximately 1 000 ft. between each pair of locks. Walking this distance and gathering in the stragglers generally required 10 min. to each chamber, so that, in all, 48 min. were taken to decompress from 40 lb. to atmosphere. No severe or fatal cases resulted, and little time was lost by the men through caisson disease, the cases being only slight. Under this pressure 330 men were employed for 36 days, working 3 hours on, 3 hours off and 3 hours on. It is true that no green men were used on this work, as there were plenty of experienced air men available at that time.

The third diagram on Fig. 6 shows the desaturation curve and rate of decompression, leaving the saturation of the blood equivalent to 27 lb. on reaching atmospheric pressure. This result bears out in part the periods of decompression of Table 4, and, if a more rapid passage had been made through the first and second locks, probably a better result would have been attained, as the equivalent pressure of the blood would have reached 25 lb. on coming out, as shown by Fig. 7, instead of 27 lb.

In caissons or tunnels with but one lock, it is a difficult problem to allow workmen as long a time as Table 4 indicates, as no one can enter the air-lock during decompression. One method of overcoming this difficulty would be to provide a lock with two small end chambers

and a larger center chamber, four doors in all being necessary. Anyone making a short visit to the caisson could pass through without disturbing the pressure in the middle decompressing chamber. Fig. 8 is a diagram of such a lock with connections.

Some one has suggested, for diving bells and caissons, a detachable chamber like a boiler which the men could enter, and thus take as long as necessary to decompress.

AIR-LOCK WITH MIDDLE DECOMPRESSING CHAMBER

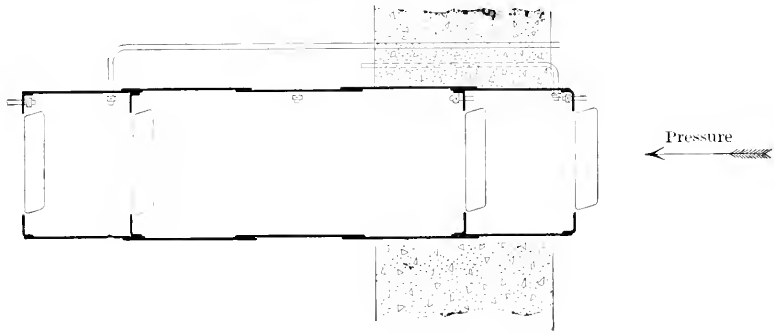


FIG. 8.

The men on the East River tunnels rebelled against 15 min. for decompression, but, after putting the responsibility up to the foremen, in time they found that it was a safeguard and voluntarily lengthened the time to 20 min., and gladly submitted to 48 min. for 40 lb.

The death rate due to caisson disease was comparatively small, averaging $\frac{19}{100}$ of 1% for the whole of the compressed-air work, and, from the experience gained, it would in all probability have been much higher if the decompression had not been lengthened.

The fact that the only recognized cure for caisson disease is recompression in a medical air-lock followed by slow decompression, is a powerful argument in favor of slow decompression, and where it is at all possible, in future works, regulated decompression will in all probability be adopted.

In caissons with small air-locks, the volume of air remaining when the lock is full of men is very slight, and very rapid decompression takes place. The workmen have a good opportunity to become seasoned in a caisson, as the pressure begins at 1 or 2 lb. and gradually increases day by day as the caisson sinks, and the highest pressures are required for but a few days.

On the East River tunnels two caissons were sunk to a final pressure of $33\frac{1}{2}$ lb. and very few cases of caisson disease occurred, none of which was fatal although the decompression was rapid. On the other hand, on account of the tunnels on the Manhattan side starting out at a high pressure, the men had no chance to get seasoned, and many cases occurred, though the time of decompression was regulated, but not to such an extent as Table 4 indicates.

Thus far, practically, no grandfatherly legislation hampers engineers in the United States in compressed-air work, but such legislation is threatened, and engineers should be prepared to guide it. The engineer is confronted with conflicting testimony on all sides, and, though his experience shows that some men are capable of rapid decompression without injury, he also knows that many men are injured. Because a few men can for a time rashly take the risk, shall he eliminate good men from compressed-air work by putting them to such a test without endeavoring to make it safe for all who are physically sound?

Prevention is better than cure, but absolute safety is impracticable, and some risk must be taken. In the writer's opinion, the hours of labor and time of decompression are matters that should receive the attention of engineers. The effect of legislation formulated by men ignorant of the conditions might greatly embarrass construction work, and, although it is contrary to the custom of this Society to draw up rules and regulations, yet it might render valuable aid if a committee of engineers, contractors, and doctors could be convened to study this subject and collect data, and ultimately draw up a table which would be a guide to the Profession. Such a table would be no doubt a compromise between the time worked in the air pressure and the time taken to decompress. It would be a statement of a body of men in high repute; if acted upon, it would relieve construction companies of liability to a great extent, and, if carefully formulated, it would greatly lessen the dangers of compressed-air work.

The writer is indebted to Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer of the East River Division of the Pennsylvania Tunnel and Terminal Railroad; and to E. W. Moir, M. Am. Soc. C. E. Vice-President of S. Pearson and Son, Inc., for permission to include such information as is taken from the work of the East River tunnels; and to Dr. Haldane for permission to make use of Table 1, and, generally, for the benefit of his theories, in writing this paper.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE SIXTH STREET VIADUCT, KANSAS CITY.
Discussion.*

By Messrs. Daniel Bontecou, and Victor H. Cochrane.

Daniel Bontecou, M. Am. Soc. C. E. (by letter).—This paper presents many points of interest, and is well worth careful reading. Mr. Bontecou

Since the author gives some details of the preliminary investigation of probable profits, it would be of interest to know more of the assumptions made as to the proportion of the total team traffic which could be depended on to use the viaduct, and of the extent to which they were realized. The estimates of cost and the operating expense given would mean that the gross earnings of the completed structure would need to be about $900 per day. The revenue from street-railway traffic could be determined fairly well, but could hardly be estimated at more than, say, $125 per day, leaving a very large proportion of the earnings to be derived from team traffic. In view of the risk in estimating the use which would be made of a utility of this kind, where it is optional to pay a toll or to take another and slightly less convenient route, it is a fair question whether so expensive a structure should have been built otherwise than by the communities served; or, whether a cheaper structure should not have been built to serve the street-car traffic only, in which case no operating expense except maintenance would be involved.

Since all the cars, and probably most of the other vehicles, must meet grades steeper than $1\frac{1}{2}\%$, it would seem that the viaduct roadway might have been placed with advantage nearer the surface, and still have afforded the proper clearances.

* This discussion (of the paper by E. E. Howard, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Bontecou. The shore line of the Missouri River at the site of the trestle on the accretion was about 600 ft. south of the structure only some twelve years before the latter was built, and has been regulated by protection work a few miles above, and also near the mouth of the Kaw. The design of the trestle foundations, therefore, involved a careful consideration of the permanence of the river regulation. The experience of the 1903 flood seems to the writer to justify the choice of pile foundations, but the conditions of soil and loading appear to call for piles longer than the concrete piles described.

In building the foundations for some large engines in similar ground near the viaduct, tests on a surface of 2 sq. ft. at a depth of about 15 ft. showed a supporting power of 1 340 lb. per sq. ft., and a settlement of 0.3 ft. under a load of 2 800 lb. per sq. ft. It was necessary to use oak piles only, 22 to 24 ft. long, and 300 were driven in an area of 2 100 sq. ft.—the intention being to depend on the combined resistance of the piles and the compressed sand, silt, and clay. After allowing for the frictional resistance of the supported concrete, the load amounted to 36 000 lb. per pile, or 4 480 lb. per sq. ft. of foundation, the value of the piles, by the Wellington formula, being 30 000 lb. This foundation was deeply submerged in 1903 without the occurrence of settlement.

In cases where piles are dependent on friction, in somewhat plastic clay, there is probably an element of safety in driving them with a heavy hammer and low fall, rather than jetting them to place, and in arranging them as far as practicable so as to confine and compact the soil. The concrete piles used in the viaduct had a surface area of 130 sq. ft., and, as placed and tested, showed a resisting power of 38 000 lb., or rather less than might be expected from piles driven in a cluster in such material, and probably less than would be desirable if the roadway were actually loaded to 100 lb. per sq. ft.

To determine the effect of clay and dirt on the strength of concrete a good many tests have been made, with varying results; and, although the writer has often permitted an admixture of 5% of earthy material, he has preferred usually to wash dirty stone in barrows having open-mesh bottoms until the water ran clear. It is difficult to understand why the presence of clay should add anything to the strength of concrete, as shown by the author's tests, and if stronger concrete can be made of well-mixed and graded materials with clay, than without it, the writer must confess to a great deal of misdirected energy in the past.

The Kaw River spans, as described, are an interesting example of riveted construction. In view of the high ratio of dead to live load, it would seem to many that a smaller impact allowance was permissible for the motorway load, and the well-understood advantages of pin connections seem to apply to this case with special aptness.

A comparison of the weight and cost of the adopted design with

one based on three pin-connected trusses for each span with cross-beams carried entirely above the top chord would be an interesting addition to the paper. Mr. Bontecou.

Ever since the flood of 1903 there has naturally been a feeling of apprehension regarding the Kaw River, and the requirement by the War Department that bridge piers be carried to bed-rock has been cheerfully accepted, regardless of the fact that, while fifteen local bridges were destroyed by that flood, there was no case in which a well-constructed pile foundation failed. It is not apparent, therefore, why it should have been thought necessary to excavate the bed-rock to receive any part of the pier construction.

Victor H. Cochrane, Assoc. M. Am. Soc. C. E. (by letter).—The writer left the office of the Consulting Engineers to take charge of the shop inspection. In some respects the work was unique, and a brief account of it, in addition to what the author has given in his unusually complete paper, may be of interest. Mr. Cochrane.

There were three inspectors on the work, two employed by the Consulting Engineers, and one by the manufacturers of the paint specified for the structure. While in the office, the writer, in the capacity of Chief Draftsman, had assisted in designing the main portion of the viaduct, and was familiar with the locality where the structure was to be built; consequently, he was well acquainted with the theoretical and practical considerations affecting the design. Accordingly, he was given authority to approve all shop drawings. As the author remarks, much time was saved by this arrangement, the drawings being put in the shops, when desired, almost immediately after being turned over to the writer for approval. The chief saving in time, however, resulted from the advice and assistance the writer was able to give to the force engaged in preparing the shop drawings. It was necessary in several instances to make considerable modifications in the design, and these changes were made without having to await the approval of the Consulting Engineers. The writer's experience in this case convinces him that the inspector in charge of a piece of work of this magnitude should be quite familiar with the design, in both its theoretical and practical aspects.

The squad of draftsmen assigned for the shop detailing seemed to consist largely of men inexperienced in making drawings for such a structure; and, although the engineers' plans were quite complete, the first shop drawings finished and checked were found to have so many errors that they had to be practically re-checked before being approved. After the men became more experienced in the work, it was not necessary to check the drawings so much in detail. Altogether, there were more than 600 sheets. From time to time, after a number of drawings had been approved, the weight of each member was computed and entered in a quadrille-ruled notebook.

Mr. Cochrane. The paint used on the work was guaranteed by the manufacturers for a period of ten years, provided they were allowed to employ inspectors to watch the work in all stages. Their representative in the shops succeeded in getting excellent work, although there was much friction at first. The paint used in assembling was so heavy and stiff that there was considerable difficulty in applying it. The painting was done under contract by Greek laborers, and, at first, it was difficult to get them to clean the metal as thoroughly as desired, since the standard adopted was much in advance of anything to which they had been accustomed; the quality of work required, however, was clearly and forcefully stated in the specifications, and, in cases of dispute, the paint inspector was nearly always sustained by the representative of the engineers.

The punching, assembling, and reaming was carefully watched throughout. Each member after being finished—but before being painted—was inspected for loose rivets and other defects, and all leading dimensions and open-hole spacings were carefully verified. In some kinds of members many pieces were nearly alike. In such cases lists were prepared and used instead of the unwieldy drawings. The dates of acceptance were noted on these lists, or on the prints in case they were used. Many pieces had to be sent back for correction, as was to be expected, but no very serious errors were found, and little material was rejected on account of surface defects. The provision for field reaming greatly facilitated the shopwork, but increased the labor of checking up field connections, as no assembling was done, and consequently there was no way to insure the matching of rivet holes except by measuring each connection.

In cases of errors the inspectors endeavored to remedy the defect in such a way as to cause the least possible expense to the shops without impairing the strength of the piece. When the shop employees learned that the inspectors could be depended on to assist them in correcting mistakes, little or no attempt was made to conceal them, and they were usually brought to the attention of the inspector as soon as discovered.

Each day the inspector entered in a scratch-book, without regard to order, the marks and number of pieces of all material inspected during the day. As soon as the invoices of shipments were received, the material listed on the invoices was checked off the scratch-book; at the same time the scale weights were compared with the calculated weights. By this means many mistakes were discovered in the invoices; some were due to misreading the marks painted on the pieces; in many cases, the scales were incorrectly read; and, in some instances, the number of members shipped was in error. The scratch-book was not intended as a permanent record, but each Saturday afternoon a list was made of unfinished work then in the shops, a column headed "Inspected" being filled out from the scratch-book. Unfinished pieces

remaining in the shops from week to week were not re-entered in this book unless advanced in construction, in which case they were checked off in a column headed "Remarks." Thus the book showed at a glance the exact state of unfinished work at any time. Mr. Cochrane.

A record of shipments was made from the corrected invoices once each week in a book, a column headed "Number Previously Shipped" giving the numbers taken from a previous record-book. Owing to the errors in the invoices, previously mentioned, the shop's records of shipments, after a time, became very unreliable, and it was necessary to depend on the inspector's records altogether.

In preparing the record-books mentioned, the headings of the various columns were not repeated on each page, but were placed near the beginning and end of the book, the tops of the intervening leaves being cut away.

The difference between the estimated and scale weights for the main viaduct was only three-tenths of 1 per cent.

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PAPERS AND DISCUSSIONS

PROGRESS REPORT OF SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE.

Discussion.*

By Wilbur J. Watson, M. Am. Soc. C. E.

Mr. Watson. Wilbur J. Watson, M. Am. Soc. C. E. (by letter).—As stated in the Minority Report of this Committee, the Majority Report, or much of it, is based on arbitrary rules for designing and executing works in reinforced concrete.

Such a set of rules should be based on sound theory, which represents the best current practice, and should be thorough and complete. It appears to the writer that these three requirements have been violated in a few important particulars. In criticizing such a report as this, it should be borne in mind, however, that we are dealing with a material, or rather a combination of materials, the use of which in engineering construction is comparatively new, and the experimental data on which, while complete enough in some ways, are very meager in others. Consequently, much confusion exists among designers with regard to the rules which they use for designing and executing in this material.

Viewed as an effort to standardize such rules and practice, the report of the Committee is commendable. A report of this kind, by a committee of this Society, is generally accepted by the majority of engineers in the United States as representing the last word on any disputed point in engineering practice; and such reports should not contain any statements which cannot be backed up with conclusive

* This discussion (of the Report printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

experimental data or rigid mathematical analysis, and, in the writer's opinion, should contain no rules which must be considered as only tentative. Mr. Watson.

In regard to the corrosion of metal reinforcement embedded in concrete, the writer is somewhat disappointed to find no reference to the danger of corrosion of such steel when used as tensile reinforcement in beams with such high working stresses that serious tension cracks occur in the surrounding concrete.

The report states that concrete is sensitive to temperature changes, but does not state to just what extent. This information is needed, and would certainly be highly appreciated. It is the writer's notion, based upon observation of numerous concrete arch structures, that concrete is not sensitive to temperature, as compared with metallic structures. In fact, this statement of the report is partially contradicted in the next article, where, under the caption "Fire-Proofing," the statement is made that "the low rate of heat conductivity of concrete is one reason of its value for fire-proofing."

The article on shear and diagonal tension appears to the writer to be open to serious criticism. The action of shear members in providing against diagonal tension due to shear stresses is now pretty well understood. It is an established principle that, when the tensile stresses in a reinforced concrete beam exceed the allowed tensile stresses in the concrete, the metal reinforcement must be depended on to take care of all the tensile stresses without any reliance on the tensile strength of the concrete.

Precisely the same reasoning applies to diagonal tensile stresses, and, to be consistent, whenever the diagonal tensile stresses exceed the allowable tension in the concrete, metallic web members should have sufficient section to take care of the entire diagonal tension. It seems to the writer that it is unnecessary to give such indefinite methods as the Committee recommends for providing against shearing stresses. Such stresses are readily computed, and the necessary web members to provide for their resistance can be easily proportioned.

In Article VIII, Section 4, a working stress of 650 lb. for concrete in compression due to bending is recommended, but no reference is made as to whether such bending stress is to be computed by the "straight-line" or "parabolic" methods, which are both in use and with results varying by about 15 per cent.

The bonding stress of 80 lb. per sq. in. allowed in Section 6 is certainly much greater than is now used by the majority of conservative designers, who do not exceed 50 to 60 lb.

In Appendix B, the symbol P is used to denote the stresses in a single reinforcing member, while the same symbol is used to denote the total safe load on a column. The writer can see no reason for reducing the area of diagonal web members to seven-tenths of that

Mr. Watson. required for vertical tension members, but recognizes the theoretical advantage of the diagonal web members.

The recognition of the desirability of continuous beam construction is to be commended, but the formulas are not those in common use, and the writer questions the wisdom of reducing the center moment below eight-tenths of that for a simple beam, as now commonly used.

On the other hand, no provision whatever is made for the increase in the shearing stresses due to continuity, and this appears to be a serious omission. The writer increases shears for continuous construction by 25% over that for simple beam construction.

This whole matter of continuity requires very careful consideration, and approximate methods of analysis should not be encouraged any more than is necessary. Most beams in reinforced concrete construction are T-beams, and the distribution of moments between the sections and the supports is far different from that in rectangular homogeneous beams, the moment at the supports being ordinarily decreased, and those in the center being increased, by reason of the T-section and the non-homogeneity of the material. However, very few designers provide sufficient reinforcement over the supports. In many cases, the writer has found that, with ordinary T-sections and percentages of reinforcement, using a center moment for uniform loads equal to $\frac{W\,l^2}{10}$, slightly more than two-thirds as much reinforcement should be provided over the supports as is provided at the centers, and care should be taken not to overstress the concrete in compression on the under side of the beam next to the supports.

It might also be well to call attention to the fact that practically all columns in reinforced concrete construction are subject to more or less severe bending stresses, owing to the fact that, while the dead load is often symmetrical about the column, the live load seldom is, and the inequality in the distribution, in monolithic construction, of the live load always produces bending moments in the columns. It would seem to the writer, therefore, that it is never safe to omit the longitudinal reinforcing rods in columns of this class.

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PAPERS AND DISCUSSIONS

STEEL SHEETING AND SHEET-PILING.

Discussion.*

By Messrs. Charles W. Sherman, C. C. Conkling, Charles H. Higgins, Charles Evan Fowler, and J. C. Meem.

Charles W. Sherman, M. Am. Soc. C. E. (by letter).—As an economical application of steel sheet-piling, the author mentions its use, in connection with dams or embankments, as a core to prevent seepage, and refers to two or three cases. While it is unquestionably true that, in original construction, such a method of making a tight core is both feasible and cheap in many cases, the writer has never been able to satisfy himself of the durability of such a core. Most embankments which serve to retain water are more or less saturated, and the steel sheet-piling, therefore, would be constantly in contact, throughout at least a part of its length, with moist earth, and rapid corrosion of the steel would seem inevitable under these circumstances. The rate of such corrosion would doubtless depend on the precise chemical composition of the steel and the character of the soil with which it was in contact. It hardly seems likely that, in ordinary cases, such a core would last longer than six or ten years, even though comparatively heavy steel sections were used. Mr. Sherman.

C. C. Conkling, M. Am. Soc. C. E. (by letter).—The writer wishes to thank Mr. Gifford for bringing out some of the practical points in the design and use of steel sheet-piling. Having taken considerable interest in the subject, during the past two years, and having discussed it in general with many engineers and contractors throughout the country, he finds that, while they nearly all recognize the merits Mr. Conkling.

*This discussion (of the paper by L. R. Gifford, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for February, 1909), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Mr. Conkling. of steel sheet-piling and its great advance over the old methods of using wood, still there is a great deal of misunderstanding regarding the requisites of a steel sheet-pile, and of the work which it has to perform during and after its installation.

Observations, covering the plans and installations of this form of piling, under many varying conditions, lead the writer to agree with Mr. Gifford that the primary quality necessary in the steel sheet-pile is its ability to resist (as a beam) earth or water pressure, either as a beam supported at the ends, as a continuous beam supported at several points, or as a cantilever beam where no bracing is used. In the first two cases it should do this with the least possible amount of bracing. In the case where the pile acts as a cantilever beam, it resists these pressures without the assistance of cross-bracing, or mainly by its resistance against bending at or near its point of penetration, and the aid which it receives by being interlocked with the adjacent piles.

This applies where the piling is used without bracing, in the construction of low single walls or circular or rectangular pockets of shallow depth. After excavation, such pockets are wholly or partly filled with concrete, clay puddle, stone, gravel, etc., in order to form units in foundations; large coffer-dams, dock walls, and other temporary or permanent constructions.

Where the piling is used as a core-wall or curtain-wall in connection with earth embankments, such as levees, dams, etc., it assists against sliding, and, in the case of dock walls, against sliding and overturning; such conditions also produce a cantilever action. In the case of walls of various kinds, the pile may also assist in giving vertical support, as a bearing pile, by friction against the earth throughout the length of its penetration. In any of these latter cases the ultimate strength of the steel wall against horizontal or oblique pressure is largely dependent on the strength of the interlocked joint of the steel piling.

Such pressures brought upon an unbraced steel sheet-pile wall, especially in its upper portion above the point of penetration, tends to bow the wall and produce tensional and transverse stresses on the interlocked joints, making the strength of these joints one of the most important factors of the steel sheet-piling.

A theoretical steel sheet-pile would seem to be one in which the strength of the interlocked joint, under tensional or transverse stress, would be equal to the strength of the web of the pile under similar stress; taking into consideration, at the same time, the lightest possible weight which will give enough metal in the pile to produce sufficient stiffness to allow it to be driven to final position, and afterward to perform its duty as a beam.

At the present time, as far as the writer has investigated, there

is no form of steel sheet-piling which exactly meets all these theoretical requirements. Mr. Conkling.

To illustrate, in part: Take a pile having a web ½ in. thick, allowing the ultimate strength of the steel in tension to be 60 000 lb. per sq. in. In a piece of piling 1 in. long, a web of this thickness would have a sectional area of ½ sq. in., and an ultimate strength in tension of 30 000 lb. The joint of two such pieces, when interlocked, should then show an ultimate tensional strength, in direct pull, of 30 000 lb. per lin. in. of joint, thereby equaling the strength of the web.

In physical tests of a great many forms of piling, by pulling to destruction in an Olsen testing machine, the writer has noted joints showing 32⅓% of the web strength, or 9 700 lb. per lin. in. of joint, as against ½ sq. in. of web, this being noted as the nearest approach to the conditions mentioned. He has also noted interlocking joints where the same test showed only 4½% of the web strength, which results indicate the great difference in the design and strength of some of the present forms of interlocking joints. These results might indicate that possibly, in some cases, the strength of the interlocking joint has been overlooked, or made secondary to supposed economy of metal, stiffness, etc.

At first thought it would seem that the thickness of the metal in the web, over and above the amount required to equalize the strength of the interlocked joint, should be eliminated, thereby getting less weight and consequent lower cost. If this were done, in some cases, it would weaken the strength of the pile as a beam, and also conflict with the author's second requirement, viz., stiffness to enable the piling to be driven without buckling.

Most demands for steel sheet-piling, at present, include lengths running from 30 to 50 ft., though engineers are now taking up propositions calling for still greater lengths, viz., 50 to 65 ft. Such increased lengths can now be manufactured and driven, thus meeting the requirements for strength, if sufficient bracing is used, spaced at proper horizontal planes so as to give safe spans for the pile as a continuous beam, supported on the different sets of waling timbers.

Experience in driving shows that it is not necessary for a steel sheet-pile to have the rigidity required in a column as commonly used in structural work. While it must be stiff enough to resist buckling under the first blows of the hammer, a slight spring under the hammer, within elastic limits, during the first penetration, is not detrimental.

The length of the pile as a column, above the point of penetration, rapidly decreases with each blow of the hammer, and, by the time the point of the pile has reached the hard, underlying formations through which the penetration is slower, the columnar length of the pile above the ground has been shortened, so that the tendency to spring under the blow has ceased.

Mr. Conkling. The entering and interlocking of the joint with that of the adjacent pile already driven tends at once to give stiffness, practically fixing the lower end. Where a slotted, cast-steel driving cap is used, being held in position by grooves against the leaders of the driver, the top of the pile is also practically fixed. These fixed conditions at the top and bottom of the piece, assist in preventing a tendency to spring under the blow.

To illustrate this point, the writer has seen a 45-ft. pile which had been bent in unloading and handling on the work so that it had a middle ordinate of about 20 in. This pile was entered and interlocked with the one previously driven, and, by careful manipulation of the pile-driver, was driven to refusal, the bend in the piece being entirely taken out as it went down by the stiffness of the members of the joint of the adjacent pile previously driven, the final result in this case being a straight pile with perfect alignment in the wall. The vertical and horizontal support which one pile receives by interlocking with another already driven is far greater than is generally supposed.

Steel sheet-piling is often used in formations, the upper portions of which are composed of clay, muck, peat, quicksand, or other soft material. A single piece of piling weighing from 1 600 to 2 000 lb., by its weight alone, will enter such material, in some cases, many feet. If it were not for the friction in the interlocking joint, the weight of the hammer resting on the pile, taken in connection with the weight of the pile itself, would be sufficient, in most cases of this kind, to produce the first 10% of penetration.

On heavy work where the lengths of the pieces run above 50 ft., producing high tonnage and demanding economy of metal, it will pay to help out the pile in its first 50% of penetration by installing a series of movable wooden guides between the leaders of the driver, which latter will prevent the tendency to spring under the blow. It would seem economical, therefore, to require no more metal in the pile than is necessary to give it the required strength as a vertical beam after it is down, and just sufficient stiffness to get it down.

For these reasons the writer would place the requirement of stiffness as third instead of second, as given by Mr. Gifford, his third requirement, "Water-tightness," being given the second place.

Under this latter heading it would be possible to enter into a long discussion on the merits of the different types of interlocking joints, as affecting the final water-tight qualities of the wall. These different types naturally fall into two general classes, namely, rigid and flexible joints. Interlocking joints of the rigid type, when in normal position, have, in most cases, very small interior clearances between their members. The chance for the material which is displaced in driving to enter and seal the portions of the joints where the steel is not in actual contact is small as compared with joints of the flexible type.

These latter, with their large clearances or pockets, allow the displaced material to fill in these spaces and be compacted by the heads of adjacent joints when they are entered and driven, thus sealing the joints so that it is not necessary to use any artificial packing. Mr. Conkling.

In driving a wall of steel sheet-piling, there are very few cases where stones in the form of large or small boulders, old logs, and other obstructions, are not struck. Sometimes these obstructions are sufficiently large, and, in the case of boulders, sufficiently hard, to prevent cutting through; especially is this condition to be met in deep work.

In such a case it is very desirable to be able, by the aid of the angularity of the flexible joint, to pass around the obstruction with the steel wall, driving to the same depth as previously done in the rear of the obstruction.

The flexible joint also allows the construction of circular or irregularly shaped pockets which are quite frequently desirable. By using joints of this type, these pockets can be built without the cost of bending the webs or additional fabrication.

In making closures in pockets, coffer-dams, or walls, the flexible joint allows distance to be gained or lost by the longitudinal displacement in the joint, or by deflecting slightly to the right or left.

These latter conditions also aid in bringing the piling back to vertical alignment, when, through meeting obstructions, or by careless driving, the top or bottom of the pile has been allowed to work ahead or back along the line of the wall. Where such inclination cannot be rectified, it may necessitate the manufacture of special tapered pieces in order to close the work.

The fixed joint, being somewhat rigid, reduces to some extent the first pressures on the waling timbers, where bracing is used in connection with the work. However, as soon as the excavation is carried to any appreciable depth, so that the pressures from the retained earth overcome the lateral stiffness of the joint, the work is thrown on the waling timbers, or, if no bracing is used, on the interlocked joints and the pile at or near the point of penetration.

It is probable that the perfect interlocking joint will finally be one which can be made flexible or rigid as desired, interlocking its members differently for each of these conditions.

Regarding the assumption that "the sections when interlocked act as a unit." By this the writer infers that the two parts of the joint are to be assumed as integral when interlocked. He does not believe this condition exists, except where the joints are grouted after the piles are driven. It is seldom that any form of steel sheet-piling is driven so that every piece making up the wall is in exact vertical alignment. Certain adjacent parts of the joints, therefore, may be in contact at any one point, and, further down in the same joint, similar points may be

Mr. Conkling. separated by a small space. Such latter spaces may be filled, at the time the pressure comes on the wall, with material displaced in driving, with silt carried in by water action, or the space may be left open. Also the small irregularities produced in fabrication and rolling tend to bring about similar conditions in the interlocked joints, all of which would prevent the two parts of the joint acting as a unit, at least in certain portions of the pile.

Therefore, if the section modulus is used as a relative comparison of the strength of the pile as a beam, it would seem better to use the section modulus of the full cross-section of one piece, or

$$\frac{\text{inertia of the section of one pile}}{\text{extreme fiber distance in same section}}.$$

The least radius of gyration, while the best comparison for theoretical stiffness as a column, should be considered in connection with the additional stiffness given to the piece by the interlock, density of material in which the pile is to be driven, and other conditions mentioned.

Economical water-tightness is certainly obtained, as explained by Mr. Gifford, by the non-requirement of auxiliary packing in the joints.

The methods generally used, to date, for specifications of steel sheet-piling, are not satisfactory, for the reasons which Mr. Gifford states.

It would seem safer to consider all pressures from saturated earth or quicksand as being hydrostatic. In cases where the wall is holding back such material, or dredged mud, it might also be better to consider the material as acting as a fluid, but having a weight of, say, 120 lb. per cu. ft., and to design the bracing to meet such conditions.

The writer would suggest that the working strength of the steel, in tension, might better be taken the same for temporary as for permanent work. Outside of deterioration of the steel in permanent work, which latter may not occur to any extent if the steel is protected, there would seem to be more liability for undeterminable pressures to be suddenly brought on the piling while in use in temporary constructions than in the more carefully planned and investigated permanent work.

Wood's formula, used by Mr. Gifford in his calculations of the strength of the pile as a beam, places the center of pressure nearer the center of the beam than is generally assumed by most authorities. This question, as well as similar ones presented in Mr. Meem's paper on the bracing of trenches and tunnels, is well worth the study and investigation of some of the expert mathematicians of the Society. There is no one line of work in which Civil Engineers are more often engaged than in building constructions to retain earth and water. If the long-used, old-time authorities are somewhat incorrect in their formulas, we cannot know it too soon for reasons of safety and economy.

With Mr. Gifford's permission, the writer would like to suggest the following additions to his specifications: Mr. Conkling.

Under "Unit Strains" he would add:

"The working stress on the extreme fiber, under any condition, shall not exceed 16 000 lb. per sq. in."

He would also add:

"*Strength of Interlocking Joint.*—The ultimate strength, in tension, of the interlocked joint of the piling, for any given unit of length, shall be not less than 25% of the ultimate strength, in tension, of the web of the pile for the same unit of length, this joint strength to be determined by a series of physical tests in a machine."

Charles H. Higgins, Assoc. M. Am. Soc. C. E. (by letter).—In this able and valuable paper, as in other discussions on the same subject, the writer has seen no reference to the value of a relatively high section modulus in preventing the bending inward of the sheet-piling below the surface of the mud, inside the dam, during excavation. Mr. Higgins.

The writer may over-value this, but it is hard to believe that his experience with this difficulty is unique, for silt such as underlies the Lower Hudson, or some material similar to it, must frequently be met in the use of steel sheet-piling.

In the particular case the writer has in mind, the sheet-piling was driven to rock through some 30 or 40 ft. of silt, the water being 8 or 10 ft. deep.

For the benefit of those who are not familiar with the character of this silt at some little distance below the surface, it may be well to explain that this material, which is exposed at low tide, or is brought up by a dredge as soft mud, is so fine as to be held in suspension in water for a considerable time. In its natural condition, a few feet below the surface, it is a plastic solid, having a rather well-defined yield point and, when not exposed to water, standing readily with a vertical face of 8 or 10 ft., which would seem to measure what has been called its yield point; this increases inversely with the degree of saturation. A 4-in. cube, now before the writer, taken more than a year ago from the river bed, retains its corners and shows no signs of crumbling.

Sheet-piling, of one of the rolled sections mentioned by Mr. Gifford, was driven into this material, the water was pumped out, heavy bracing was placed, excavation was begun, and all went well. The bracing was placed immediately after removing about 6 ft. of silt, other bracing being placed every 3 ft. until a depth of some 15 ft. below the mud line, or from 20 to 25 ft. below the water line, was reached. Below this point difficulty was experienced in placing the braces, due to the sheet-piling bending inward below the line of excavation. This difficulty increased with the depth, and, although powerful

Mr. Higgins. hydraulic jacks were used, it became impossible to force the steel back to its original position; the area of the dam became more and more contracted, until at a depth of about 35 ft. below the water line, near the point where the rock was highest, the foot of the dam kicked in, and mud and water followed rapidly. The coffer-dam filled, and was only emptied and the excavation completed to rock by decking the entire structure and continuing the work under compressed air.

This sheet-piling had a radius of gyration of 0.8 and a section modulus of about 4.0 per ft. of width; it stood driving, even through old timber, behaved well with the bracing in place, and failed utterly through insufficient stiffness to withstand the unbalanced pressure in this plastic solid or cheese-like material below the line of excavation.

As far as the writer knows, this point has not yet been raised, although under certain conditions it would seem to be the determining feature in the choice of a steel sheet-piling, or even as to whether it might be advisable to use any form of an open coffer-dam. The writer calls attention to it in the hope of bringing forth some further information.

The huge coffer-dam of steel sheet-piling at Buffalo, mentioned as under construction, has recently been pumped out, and all interested in coffer-dams would be glad to learn what degree of success attended this operation.

Mr. Fowler. Charles Evan Fowler, M. Am. Soc. C. E. (by letter).—The use of metal sheet-piling is very limited in the Northwest, where the writer is located, and his opportunities for observing its use have been very infrequent.

When the writer discovered the paper by Mr. Borthwick, published in 1836 in the *Minutes of Proceedings* of the Institution of Civil Engineers, he was very much surprised that metal sheet-piling had not been used more extensively in the past. It is especially remarkable that, upon the extended use of wrought iron and steel in place of cast iron, metal sheet-piles were not used more generally, but the reason, doubtless, has been that, until of late years, timber for the construction of coffer-dams has been remarkably easy to obtain and has been of low cost, as compared with wrought iron and steel.

In a great many cases it is also true that for such temporary construction timber is better adapted than steel in any form, mainly owing to the fact that it is necessary to make connections of bracing and staging at various points, and these connections can be made more easily with timber than with metal. In view of this, it is doubtful whether metal will ever supersede entirely the use of timber for sheet-piling.

A great point in favor of metal, as compared with timber, is the fact that, with the various dove-tailed patterns in use, the piling can

be driven more closely and made more nearly water-tight than with any form of timber sheet-piling, even where a tongue and groove of dove-tailed form has been spiked to the timber, as this is apt to pull off in driving and leave bad leaks in the timber wall. Mr. Fowler.

With reference to the pressures on sheet-piling, the author has evidently overlooked the fact that in a very great number of cases the pressure is not that due to the water, but to that from semi-liquid or very soft mud, and may easily run up to 50% more than that of water.

The specifications for metal sheet-piling proposed by the author are certainly satisfactory as to the class of steel to be used, as there is no need of having it manufactured under any special specifications. In the matter of unit stresses for steel, it would be safe to use a fiber stress of 24 000 lb. per sq. in. in the temporary work, and 20 000 lb. for permanent work. The ratio of length to least radius of gyration is certainly high enough, but this is a matter which can be determined very largely from experience obtained by actual use of the various forms.

In the large majority of cases, the types shown in Group 1 would doubtless prove most satisfactory, owing to their greater sidewise stiffness and the better opportunities for making connections to them. There are, however, a great many cases, especially in shallow work, where the types shown in Groups 2 and 3 would be more satisfactory, on account of their lightness and less cost. Where the work is of any magnitude, it is always best to make a thorough investigation for each case, and adopt the form which is most satisfactory for the particular location.

J. C. Meem, M. Am. Soc. C. E.—The speaker is a strong believer in interlocking steel sheet-piling, and, while he does not wish to appear as an advocate of any special type, he is glad to take this opportunity of speaking a word in favor of the more general use of this material. Mr. Meem.

Reinforced concrete piling is subject to the same fundamental disadvantages as those debited against wooden sheet-piling, in that it does not interlock and therefore may be forced apart by obstructions encountered in driving; and, further, that the corners cannot be made secure against an influx of sand or other material during excavating and pumping. Further, the speaker does not believe that the fear that harm may result from steel piling left in the ground is well founded. He bases this assumption on his belief that piling driven into the ground under water will not deteriorate after the outside heavy protective coating of rust has established itself, owing to the fact that there will be little or no further opportunity for oxidation, as long as the piling is not actually exposed to free water; and,

Mr. Meem. further than this, if the steel should rust out entirely, the space occupied by the rust or oxidized iron ought to be far in excess of that of the steel itself, leaving no opportunity for voids to occur.

The speaker believes that the use of interlocking steel piling in long lengths is too well established as an engineering factor to make it necessary to point out its advantages; but he is not so sure that it is as well-known a fact that this material can be used to great advantage at times in very short lengths. He has had occasion to use very short steel piling in connection with the underpinning of buildings and existing subways along the line of construction of new work, and the following is a brief description of the methods used:

The piers of the building or subway to be underpinned were first shored on temporary beams in the ordinary way; 6 by 6-ft. pits were then excavated and box-sheeted with ordinary 2-in. planking down to the level of the water, after which 4 by 4-ft. frames of interlocking steel piling of the "United States" type, usually in 6-ft. lengths, that is, sufficiently long to reach about 2 ft. below the sub-grade of the adjacent excavation, were set up and driven to full depth; after which the pits were mucked out to within a foot of their bottom and filled with concrete, the processes of excavation and concreting being carried on without unwatering the pile pit.

The result of this was that there was no influx of sand under the piling, as would have been the case had the pit been unwatered without having driven the piling to a much greater depth below excavation. As soon as the concrete in the pile pit had set sufficiently, the remainder of the pit was filled with concrete or bracing, and the work of underpinning was completed in the usual manner.

It is interesting to note that in many of these footings, which were nothing more than large square piles of from 1 to 2 ft. penetration, there was never any evidence of settlement, although they were at times subjected to a pressure of as much as 25 tons per sq. ft., with the adjacent material excavated to within at least 18 in. of the toe of the steel piling.

It is sometimes desired to use steel sheet-piling in very much longer lengths than can be handled conveniently owing to lack of head-room in building coffer-dams under streets and in congested places. In such instances it may be advisable to cut the piling into shorter lengths, driving the second and third sets as followers. Care should be taken, however, to break joints between the adjacent piles, and splice-plates should be tap-bolted into the ends of the abutting sections in order to keep one pile from crawling away from its follower while driving the adjacent member. Circular frames, 5 ft. in diameter, of steel piling in lengths of about 6 ft., have also been used to good advantage by the speaker as sumps for pumping out coffer-dams. Sumps of this shape can be driven continuously without the use of bracing, and are

very effective. Piling of the "United States" type in 6-in. widths has been used most extensively by the speaker in these operations, but piling of the "Lackawanna" type or any flexible type should be equally effective. Great care should be taken in the selection of steel piling to secure the type best suited to the conditions. Mr. Meem.

It has come to the speaker's notice that a coffer-dam designed to have its piling act as a vertical cantilever resulted in developing stresses similar to those found in a suspended chain stressed horizontally. This condition, fortunately, had been foreseen by the engineer in charge for the contractor, and he selected for use a piling which gave a high strength of cross-pull in the interlock. In relation to this matter Table 1 gives some figures which show a wide variation in cross-pull at the interlock, as found in the "Freistadt," the "United States" and other types. While the "Freistadt" is particularly effective as a beam, it has very little resistance to cross-pull in the interlock. The speaker is indebted to the Carnegie Steel Company for the figures in Table 1 relative to the "Freistadt" and "United States" types, and to the engineers of the Lackawanna Steel Company for the figures relating to the "Lackawanna" type. It is only fair to say, however, that a built-up section such as suggested by Mr. Skinner should also give good results, if specially designed for effective resistance against cross-pull.

TABLE 1.

Type of piling.	Section of piling.	Resistance to cross-pull per linear inch at interlock.
"Freistadt" steel sheet-piling.	15-in. by 35-lb. channels, and 4-in. by 3/8-in. zees	1 500 lb. per lin. in.
	15-in. by 50-lb. channels, and 5-in. by 1/2-in. zees	2 250 " " " "
"United States" steel piling	12-in. by 40-lb.	12 000 " " " "
"Lackawanna" steel piling	12¾-in. by 1/2-in. web	9 970 " " " "
	12¾ in. by 3/8-in. web	9 970 " " " "

It would appear from the figures in Table 1 that very great advantage may be derived at times from designing coffer-dams so that the material in them will give thrust developing these stresses in the piling, thus saving a large amount of cross-bracing. These stresses can best be developed by driving the piling between supports with a "belly" or bow, making the versed sine of the latter as great as possible with reference to the horizontal distance.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DANIEL DAWSON CAROTHERS, M. Am. Soc. C. E.*

DIED JANUARY 2D, 1909.

Daniel Dawson Carothers, the fifth son and seventh child of Rezin Dawson and Elizabeth Bain Carothers, was born at Cutler, Washington County, Ohio, on August 21st, 1860.

In early life, his father had been a millwright, but later he became a railroad contractor. He had a fine mechanical turn of mind, which his son inherited, and was noted for his determination and energy. His ancestors had come to America from Scotland, in Colonial days, and had settled in Huntington County, Pennsylvania. Mrs. Rezin Carothers was of English extraction, her ancestors having emigrated from England to Maryland, subsequently moving to Beaver County, Pennsylvania.

The early life of Daniel Dawson Carothers was passed on a farm, where he was required to do the "chores" and odd bits of work which fall to the lot of a country boy. This early experience tended to develop both mind and body, and the boy showed a strong liking for work with tools and for everything which pertained to machinery. His mother exerted a strong influence over his early life and intellectual development, giving him his primary schooling, as the neighboring country schools, in those days, were poor and the term of study very short. The last year or two of Mr. Carothers' school life was spent at Bartlett Academy; later, he went to the National Normal University at Lebanon, Ohio, where he took a special course in engineering, although he did not enter for a degree. During the last three years of Mr. Carothers' college life, he taught school during the winter months. Finally, in 1882, he began his labors in the field which he had chosen for his life work. At that time his father was connected with the Wheeling and Lake Erie Railroad, and the young civil engineer was given employment as a Rodman under his father, who early impressed upon him, as the primary necessity for success in business, the spirit which would "always obey orders."

As Rodman and Assistant Engineer, Mr. Carothers was with the Wheeling and Lake Erie Railroad during 1882. The next year he entered the service of the Columbus and Cincinnati Midland Railroad (now a part of the Baltimore and Ohio System), serving as Assistant

* Memoir prepared by Paul Didier, M. Am. Soc. C. E.

Engineer, Chief Engineer, and Train Master. He remained with this Company for seven years, or until 1890, when he transferred the field of his activity to the Baltimore and Ohio Railroad, with which road, or its subsidiary lines, he remained until his death.

When Mr. Carothers entered the service of the Baltimore and Ohio in 1890, it was as Engineer of Maintenance of Way of the Baltimore and Ohio Southwestern, which position he held until 1901. He then became Superintendent of the Chicago Division of the Baltimore and Ohio Railroad, with headquarters at Chicago; two years later, in 1903, he was made General Superintendent of the Baltimore and Ohio Southwestern, with headquarters at Cincinnati. In 1904 he was chosen Chief Engineer of the Baltimore and Ohio Railroad, with headquarters at Baltimore, which position he held at the time of his death.

Mr. Carothers was married on September 20th, 1888, to Miss Carrie Leland of Lewiston, Maine. They had one child, a son, who lived only a short time. Mr. and Mrs. Carothers resided in Baltimore for the past five years, and made many friends. Mr. Carothers was a Member of the Maryland, Merchants, Baltimore Yacht, Baltimore Country, and Engineers' Clubs of Baltimore; the St. Louis Railway Club; the Western Railway Club of Chicago; the American Railway Association; the National Geographic Society; the American Railway Engineering and Maintenance-of-Way Association, of which he was a Director and Chairman of the Rail Committee; a Member of the Committee on Conservation of National Resources, having attended the two conferences of the Governors of the States with the President of the United States, at Washington; the Ohio Society of New York; and the Madisonville Lodge of Masons.

Entering the service of the Baltimore and Ohio Railroad on the eve of a new era in railroading in America, while the many companies were beginning such marvelous strides in the way of development, Mr. Carothers played an important part in the upbuilding of one of the greatest railroad systems in the country. During the years he filled the many important positions to which he was chosen prior to 1904, he was instrumental in directing much of the energy then being devoted to railroad improvement.

Mr. Carothers was a man of pleasing personality, and typified, not only a success as an engineer, but a success as a man. As an engineer and railway official, he had initiative power, and his education and large experience, backed up by one of his most dominant characteristics—common sense—gave great strength to his views among other men of his Profession. As a man, his sterling qualities and manliness drew everyone to him. The men who worked in close touch with him were those who most appreciated and respected his opinions. Notwithstanding the position in the railway world which he had made for himself, those who knew him felt that this was but the

beginning of his true usefulness, and all experienced a deep sense of personal loss at his death.

"The world was better because he lived. A noble spirit has gone to rest."

Mr. Carothers was elected a Member of the American Society of Civil Engineers on April 4th, 1904.

RUTGER BLEECKER GREEN, M. Am. Soc. C. E.*

Died December 8th, 1908.

Rutger Bleecker Green was born in Syracuse, New York, on October 8th, 1868, and was the second son of Andrew Heatley Green, a prominent attorney of that city, and Mary Miller Green. He prepared for college in the public schools of Syracuse, New York, and Morristown, New Jersey, and entered Cornell University in 1890. He was graduated with the Class of 1895, having lost one year because of sickness. As an expression of their appreciation of his ability and of their regard for him, his classmates, during his Senior year, bestowed upon him the highest honor within their power by electing him Chief Engineer of the survey made by the Senior and Junior classes at the northern end of Owasco Lake, New York.

Previous to entering college Mr. Green had had considerable practical experience, having been engaged as Chainman on the New York State Canals; Chainman and Instrumentman on the Richmond and Danville Railroad; Chainman on the Adirondack and St. Lawrence Railroad; and Assistant Engineer on the construction of the Syracuse Water-Works' pipe line from Skaneateles Lake, being in charge of a cross-section and construction survey party, on Sections 1 and 3, in 1893.

After graduation, Mr. Green was an Assistant on the editorial staff of *The Engineering Record* for nearly two years. He left *The Record* to accept the position of Leveler on the New York State Canals. In November, 1898, he accepted a position as Civil Engineer with the Detroit Works of the Solvay Process Company, in which position he remained until his death. He had charge of the land surveys of the 400-acre plant on the Detroit River; designed and constructed a yard system of 8 miles of tracks, with 75 turn-outs and an 8-stall locomotive house, 1 mile of concrete sewers, 2 miles of gas and water pipe for manufacturing and fire protection, including a yard system of 40 hydrants, and automatic sprinkler systems. He also supervised the construction of the foundations and superstructure of a 300-ft. railroad draw-bridge and coal-car tipple, and the enlargement of the

* Memoir prepared by M. S. MacDiarmid, Assoc. M. Am. Soc. C. E.

foundations of a 260-ft. chimney, also the foundations of various office and factory buildings, including lime-kilns, dynamo-house, ammonia tanks, concrete bulk-soda storehouse and office building.

Personally, Mr. Green's disposition was particularly loving and sympathetic, and all with whom he came in contact were pleased to call him friend.

Special mention should be made of his interest in religious and philanthropic affairs, as at the time of his death he was Treasurer of St. Mark's Episcopal Church at Delray, and a Trustee of D'Arcambal Home.

Too close application to his work, and the intense interest which he took in everything in which he engaged, resulted in a nervous breakdown. In June, 1907, Mr. Green took a trip to Europe for his health, but he was not permanently benefited.

A recurrence of the old trouble during the summer and fall of 1908 caused such a state of mental depression that, with the words, "I am worked out," he sought rest in death, on December 8th, 1908.

Mr. Green was elected a Junior of the American Society of Civil Engineers on May 5th, 1896; an Associate Member on October 5th, 1898; and a Member on September 6th, 1904. He was elected a Member of the Detroit Engineering Society on October 26th, 1900. He contributed valuable discussions to each of these Societies.

CHARLES MacRITCHIE, M. Am. Soc. C. E.*

Died January 27th, 1909.

Charles MacRitchie was born in Inverness, Scotland, in 1844. From 1860 to 1865, he served as an apprentice with G. E. Mackay, Land Surveyor and Civil Engineer, of Inverness, Scotland.

Mr. MacRitchie began his professional work in 1866, in the office of Edward Blythe, of Edinburgh, Scotland, being employed on the construction of the Dingwall and Skye Railway. Part of the time he acted as Resident Engineer on the western half of the road.

In 1869, Mr. MacRitchie came to the United States and, after a short stay in New York, went to Chicago, where he was employed in the private office of the late E. S. Chesbrough, M. Am. Soc. C. E. (who, at that time, was City Engineer), in making plans for a railway tunnel under the Detroit River, at Detroit, Michigan.

Under the direction of and in association with Mr. Chesbrough and the late Moses Lane, M. Am. Soc. C. E., Mr. MacRitchie made surveys and plans for the water supply of Pittsburg, Pennsylvania, and Long Island City, New York. He was also Principal Assistant Engineer

* Memoir prepared by John Nichol, M. Am. Soc. C. E.

on the construction of the water-works for Milwaukee, Wisconsin, and afterward served as Secretary of the Board of Trustees.

In 1875, Mr. MacRitchie entered into partnership with John Nichol, M. Am. Soc. C. E., under the firm name of MacRitchie and Nichol, Engineers and Contractors, which partnership continued until 1906. This firm prepared plans and contracts and carried out work, either for a complete water supply, or a part thereof, in many cities in the United States, which included some interesting submerged-pipe work in Galveston, Texas, Boston, Massachusetts, and St. Paul and Minneapolis, Minnesota, and also some railway-construction work.

From 1906 until his death, Mr. MacRitchie was in partnership with Mr. Benezette Williams, engaged in railway and other contracting work.

By his genial, kindly and generous disposition and straightforward honesty of purpose, Mr. MacRitchie made many friends wherever he went.

Mr. MacRitchie was elected an Associate of the American Society of Civil Engineers on October 2d, 1872, and a Member on April 5th, 1876.

CHARLES TARBELL DUDLEY, Assoc. M. Am. Soc. C. E.*

Died September 30th, 1908.

Charles Tarbell Dudley was born in Richmond, Indiana, on April 6th, 1878. He received his technical training at Sheffield Scientific School of Yale University and at the Colorado State School of Mines. He was graduated from the former with the degree of Ph. B. in 1900, and from the latter with the degree of E. M. in 1902.

While a deep student and an investigator along original lines, his memory is particularly lasting at these two schools because of his tireless efforts in the up-building of clean, wholesome, college athletics. Though small, Mr. Dudley was an athlete of the wiry, determined type, and Harvard and Princeton still remember him as a most dangerous half-back, centerfielder, and batsman. Determination and a high moral standard were his chief characteristics, and these, coupled with a cheerful, lovable disposition, endeared him to all his associates.

Before taking up his post-graduate work at the Colorado State School of Mines, Mr. Dudley secured employment as Assistant Chemist at the Midvale Steel Works. In six months his ability was recognized to such an extent that he was put in charge of the Department of Metallography and Experimental Chemistry, and in another six months he had invented a new process for the treatment of large, nickel-steel

* Memoir prepared by H. L. Haehl and C. E. Gilman, Assoc. Members, Am. Soc. C. E.

forgings, which nearly eliminated the imperfections so frequent before that time.

After his post-graduate course, Mr. Dudley went to California and acted as Superintendent of various mining properties until September, 1903, when he accepted employment with the Engineering Department of the Bay Cities Water Company, which corporation was at that time designing a new water supply for the City of San Francisco.

In 1905 Mr. Dudley resided in the Santa Cruz Mountains, California, and designed the trestles and bridges for the Santa Cruz Portland Cement Company's railroad. The great California earthquake of 1906 interrupted this work, and, seeing the opportunity for designing and structural engineers in the rebuilding of San Francisco, Mr. Dudley moved to that city. He was engaged in the design of some of the largest of San Francisco's new buildings when he was seized with an attack of typhoid fever, and for six weeks he fought for his life with his characteristic determination. Before he had regained his strength he moved to Allston, Massachusetts, and began again the practice of his profession. His strong constitution, in its weakened condition, could not stand this added strain, and a general breakdown resulted, against which his determination and grit were unavailing. Mr. Dudley died on September 30th, 1908. His wife, Sarah Emery Dudley, whom he married in Boston, in 1905, and two children, Sarah and Wade, survive him.

To those who knew him his death was a keen blow; to his friends, it was a great loss—the loss of that great possession, a wholesome, manly man and a true friend.

Mr. Dudley was elected a Junior of the American Society of Civil Engineers on September 6th, 1904, and was transferred to the grade of Associate Member on October 3d, 1906.